Behavior of Contraction Joints in the Rehabilitated AASHO Test Road

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16. Abstract

Rehabilitating the AASHO Test Road pavement gave an opportunity to examine the behavior of sawed dowelled contraction joints spaced at 15 ft (4.57 m) in non-reinforced pavement overlying a granular subbase with those sawed at 40 ft (12.19 m) and at 100 ft (30.48 m) in reinforced pavements on both granular and stabilized subbases. Behavior for this study represents a change in spalling, faulting, joint opening, D-cracking, transverse cracking and pavement smoothness. Faulting decreased as joint interval decreased and as pavement thickness increased. Faulting was reduced where the subbase was stabilized. The cumulative amount of faulting per pavement mile was largest for 15-ft panels even though they had the least fault per joint. This fact partly accounts for pavements with 40-ft joints being smoother than those with 15-ft joints. The amount of spalling per mile of pavement increased as the joint interval decreased, although the number of major spalls per joint tended to increase as joint interval and joint opening increased. Transverse cracking between the joints increased as joint interval increased, but it was reduced over a stabilized subbase. The amount of D-cracking per mile of pavement increased as the number of joints and cracks increased and as the pavement aged. The best overall pavement behavior and the lowest Roughness Index were associated with pavements that had the fewest joints, particularly on a BAM subbase.

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Department Of Transportation
DIVISION OF HIGHWAYS
Bureau of Materials and Physical Research

BEHAVIOR OF CONTRACTION JOINTS IN THE REHABILITATED AASHO TEST ROAD

By

L. J. McKenzie, R. J. Little, and P. G. Dierstein

Interim Report
IHR-28

AASHO ROAD TEST - Phase 1

A Research Study Conducted by
Illinois Department of Transportation
Springfield, Illinois 62706
in cooperation with
U. S. Department of Transportation
Federal Highway Administration

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INTRODUCTION

In 1962, when the AASHO Test Road near Ottawa, Illinois, (Figure 1) was converted into a part of Interstate 80, a number of experimental sections of portland cement concrete pavement were built to replace those test sections that had failed or that did not meet the requirements for an interstate highway and to connect the test tangents into a continuous roadway through the test area. This arrangement permitted a comparison of the behavior of the Illinois standard pavement with that of the surviving AASHO pavement designs under the same environmental conditions while serving normal mixed highway traffic.

Sawed, dowelled, transverse contraction joints were used in all the AASHO pavement sections as well as in all the sections constructed in 1962. They were spaced at 15 ft (4.57 m) and at 40 ft (12.19 m) in the AASHO pavement sections, and at 40 ft (12.19 m) and at 100 ft (30.48 m) in the new pavement sections.

The objective of this study was to investigate the behavior of the sawed, dowelled, transverse contraction joints as related to the three joint intervals that had been used and as related to the granular, BAM and CAM subbases on which the pavements were placed (1).

The data gathered for the investigation span 12 years, from November 1962 when the rehabilitated roadway was opened as part of Interstate 80, until 1974, and include periodic inventories of pavement cracking, D-cracking, spalling, joint faulting, winter joint openings, and pavement roughness. Prior to resurfacing in 1975, two 4-ft pavement sections containing typical joints were removed from the roadway and were examined for dowel bar corrosion, joint lockup, and concrete condition.

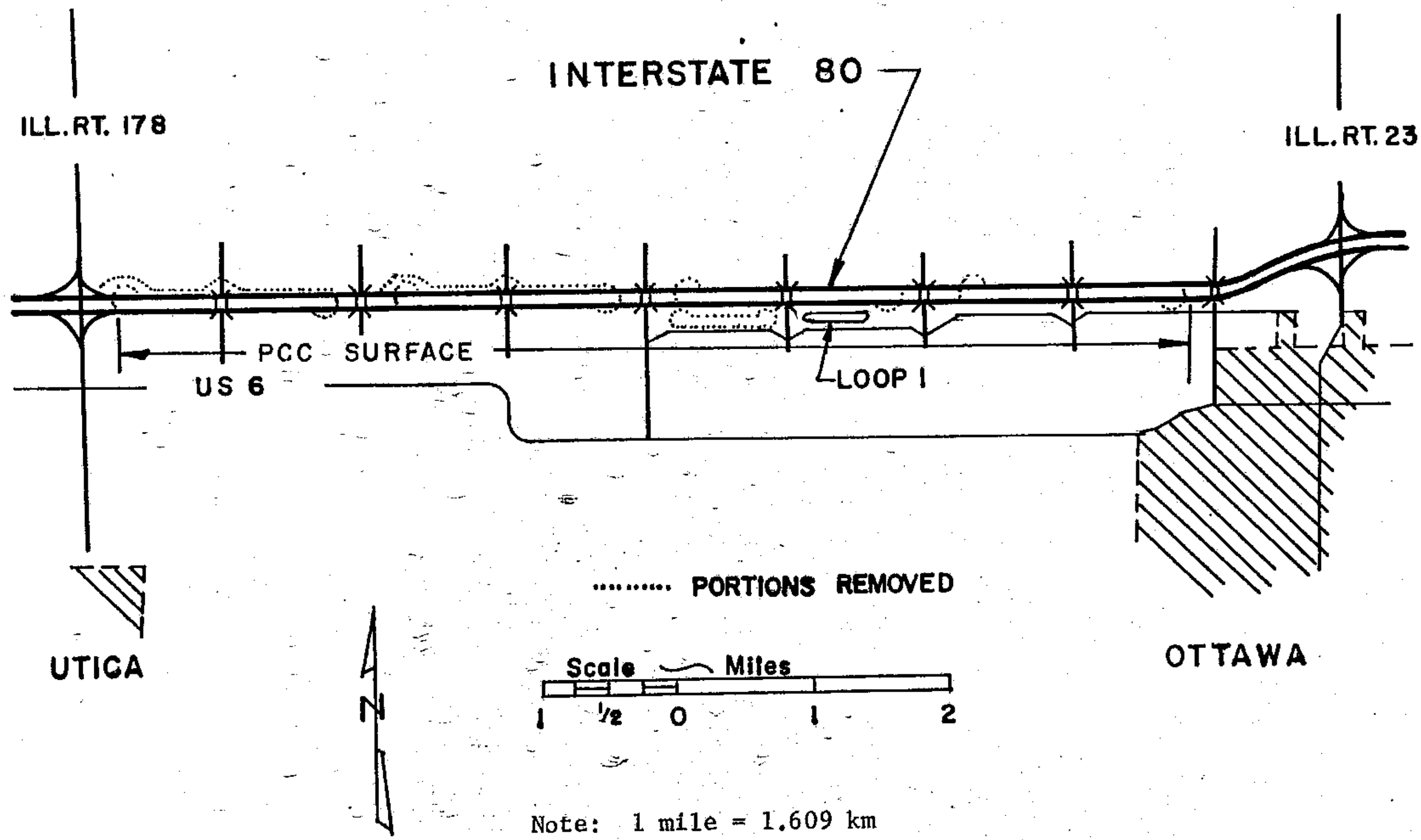


Figure 1. Layout of the Rehabilitated AASHO Test Road.

The inventory data indicated that faulting decreased as joint interval decreased and as pavement thickness increased. Faulting was reduced where the subbase was stabilized. The cumulative amount of faulting per pavement mile was largest for 15-ft panels even though they had the least fault per joint. This fact partly accounts for pavements with 40-ft joints being smoother than those with 15-ft joints. The amount of spalling per mile of pavement increased as the joint interval decreased, although the number of major spalls per joint tended to increase as joint interval and joint opening increased. Transverse cracking between the joints increased as joint interval increased, but it was reduced over a stabilized subbase. The amount of D-cracking per mile of pavement increased as the number of joints and cracks increased and as the pavement aged. The best overall pavement behavior and the lowest Roughness Index were associated with pavements that had the fewest joints, particularly on a BAM subbase.

In the two pavement joints removed, dowel bar corrosion was found to be the cause of joint lockup. The forces required to free the joints were sufficiently large that joint lockup may have been an important factor in the formation of the major cracks.

This report briefly outlines the development of the present joint design standards, enumerates experimental details, describes the behavior of the pavements associated with 15-ft, 40-ft and 100-ft joints and evaluates the effect of the various subbase types on joint behavior. The examination of two pavement sections that contained typical sawed, dowelled, transverse joints also is described. The implications of the various forms of pavement and transverse joint deterioration are discussed, conclusions are drawn, and recommendations are made.

BACKGROUND

Illinois' standard design for pavement joints has evolved from none in the early 1920's to an exclusive requirement for sawed, dowelled, transverse contraction joints spaced at 100 ft (30.48 m). Because of this long spacing, the pavement was reinforced with welded wire fabric to keep expected transverse cracks tight, and load-transfer dowels were placed in the joint to reduce faulting. Until just recently, only minor changes have been made in the standards. Among the changes have been an increase in dowel bar diameter and an adjustment in the size and weight of the wire mesh reinforcement to conform with variations in slab thickness.

When the Division of Highways began paving with portland cement concrete, Illinois engineers expected that thermal pavement length changes would be absorbed in horizontal curves and that transverse pavement cracks could be easily sealed. However, experience proved otherwise, and the repair of pavement blowups became a serious and a costly maintenance problem (2, 3).

As a solution to the problem, one highway district installed 4-in. expansion joints at approximately 1/2-mi intervals. This practice apparently achieved some degree of success because use of the 4-in. expansion joint soon became widespread in the State, and by 1928 it was adopted as standard practice. Spacing of the expansion joints gradually decreased until 1928 when the standards required that they be placed at not less than 800 ft (243.84 m) nor more than 1000 ft (304.80 m). After a number of miles of pavement that incorporated the 4-in. open expansion joints were built, it became apparent that the joints were closing rapidly. Bituminous filler was extruded from the joints and formed ridges which had to be removed several times a year. Then, after the pavement contracted, the filler had to be replaced. Incompressible material that found its way into open cracks caused the joints to close at a rate which averaged about 1 in. per year, At this rate, pavement blowups were expected during the fifth year, but some occurred even sooner (2, 3).

After comprehensive study of the problem and after several trial installations, the Division, in 1933, specified the construction of copper-sealed, all-metal, air-chamber expansion joints providing a 3/4-in. to 1-in. expansion space. They were installed every 90 ft (27.43 m). To control the formation of pavement cracks between the expansion joints, copper-sealed, metal contraction joints were placed 30 ft (9.14 m) from each expansion joint. The use of dowel bars or some other approved form of mechanical load transfer in each joint also was specified. By 1937, serious defects were showing up in the metal joints. The copper seals failed, permitting the entrance of water and incompressible material. The concrete at the joint faces deteriorated and transverse cracks still appeared in some panels. Surprisingly, the frequency of blowups was reduced. Therefore, in 1938, the Division introduced a 3/4-in. to 1-in. expansion joint spaced at 50 ft (15.24 m). The joint contained a pre-molded filler of fiber, rubber, cork, or cork-rubber and some approved form of mechanical load transfer. Knowing that transverse cracks would occur, welded wire fabric reinforcement was required to keep them tight.

During World War II, (1942-1945) the use of steel in pavement construction was discontinued to aid in the war effort, but its use was resumed as soon as steel became plentiful again. During the steel shortage, plain concrete pavements were built with dummy grooved contraction joints every 20 ft (6.10 m) and expansion joints every 120 ft (36.58 m).

Then, conforming with a nationwide trend in 1946, Illinois dropped the use of expansion joints in mainline pavements and adopted full-depth metal contraction joints spaced at 100 ft (30.48 m). Expansion joints were no longer required except in special places such as adjacent to bridge approach slabs and at intersections.

In the early 1950's when it became apparent that the concrete was deteriorating at the metal joint face, a sawed joint was introduced as an alternative. By 1955, the metal plate was dropped completely in favor of sawed joints. They were spaced at 100 ft (30.48 m) and contained load-transfer dowels placed 13½ in. (342.9 mm) center-to-center. Wire mesh reinforcement was required in the pavement slab.

When the AASHO Test Road was built during the late 1950's (Figure 1), sawed transverse contraction joints were used throughout the rigid pavement experiment. The non-reinforced sections had joints at 15 ft (4.57 m) and the reinforced sections had joints at 40 ft (12.12 m). All reinforcement was wire mesh which varied with pavement thickness as can be seen in Figure 3. Dowels, spaced 12 in. (304.8 mm) apart, were placed in all joints for load transfer.

When the AASHO Road Test concluded in 1961, the experimental loops were rehabilitated to form a four-lane divided highway (Interstate 80) through the test area. During rehabilitation most of the original rigid pavement sections, 8 in. (203.2 mm) and greater in thickness, which had not been damaged during the testing, were retained as part of the new roadway. All others were removed.

Except for a few test sections that were rebuilt as replicates of original designs or that represented new designs in the test tangents, the new connecting pavements were 10 in. (254.0 mm) thick. They were reinforced with welded wire fabric, and they contained sawed, dowelled, transverse contraction joints spaced at 100 ft (30.48 m). Some of the new 10-in. pavement was included in the experimental regiment as test sections. Therefore, the rehabilitated test road contained test sections with sawed, dowelled, transverse contraction joints spaced at 15 ft (4.57 m), 40 ft (12.12 m), and 100 ft (30.48 m).

The pavements have been in regular service since 1962, and during that time almost 26 million vehicles have used the roadway. Periodic inventories of road smoothness, pavement cracking, and joint spalling and faulting have been made. These data have been combined to study the relationship between transverse contraction joint behavior and joint interval. This report presents the comparisons and the findings that have been made from the analyses.

EXPERIMENTAL DETAILS

The rigid pavement test sections are located in the eastbound lanes of the experimental highway, and only the test sections in the outside traffic lane were used in this investigation. Rehabilitating the AASHO Test Road as part of Interstate 80 is described in detail in Illinois Department of Transportation Physical Research Report No. 51 (1), which includes a listing and a layout of all the experimental pavements in the rigid pavement experiment. The AASHO Road Test is described in detail in six Highway Research Board Special Reports (4-9). Detailed descriptions of the experimental pavement sections from the original test road that were retained as part of Interstate 80 and have yielded data for this report are contained in HRB Special Report No. 61B (5).

Climate

The climate at the AASHO test road site is temperate. The mean annual summer temperature is 76°F (24°C) and the mean winter temperature is 27°F (-3°C). Although the coldest months are December, January and February, mid-day temperatures often exceed 32°F (0°C).

Average frost penetration is 28 in. (710 mm). Frost enters the soil below the pavement about mid-December and disappears in early March, but light surface freezing occurs both earlier and later. Occasionally, frost leaves the soil completely for short periods during January and February.

Precipitation averages 34 in. (860 mm) annually and falls mostly as rain during summer thundershowers. Snowfall amounts to about 25 in. (630 mm), but snow cover does not regularly persist for more than a few days. During the winter, there are 8 to 9 days on the average when icing makes highway travel hazardous.

Traffic

Because the test facility is located between two interchanges, every vehicle traveling eastbound must pass over each rigid test section; hence, all the rigid test sections have received the same number of load applications since 1962. The average daily traffic and the average daily commercial traffic through 1974 are plotted in Figure 2. Average daily commercial traffic comprises 21 percent single units and 79 percent multiple units.

In addition to the regular mixed highway traffic the test sections from the AASHO Test Road have carried controlled axle loads applied during the original test period. To resolve the differences between the AASHO test traffic and mixed highway traffic, all traffic was adjusted to equivalent 18-kip single-axle load applications (ESAL'S) using the equivalency factors developed from the AASHO Road Test data (10). Because each test loop in the AASHO Test Road carried a different axle weight, the 18-kip ESAL'S for the AASHO test sections differ by test loop. The cumulative numbers of 18-kip ESAL'S through 1974 for rigid test sections in the rehabilitated test road are tabulated below:

Test Group	18-Kip ESAL'S (Millions)
New Sections (1962)	10.00
AASHO Test Sections, Loop 4	11.64
AASHO Test Sections, Loop 5	14.09
AASHO Test Sections, Loop 6	18.64

Conversion: 1 kip = 453.6 kg

Test Sections

The rehabilitated test road now has 84 rigid test sections in the mainline pavement, of which 47 are AASHO test sections from Loops 4, 5 and 6. The AASHO

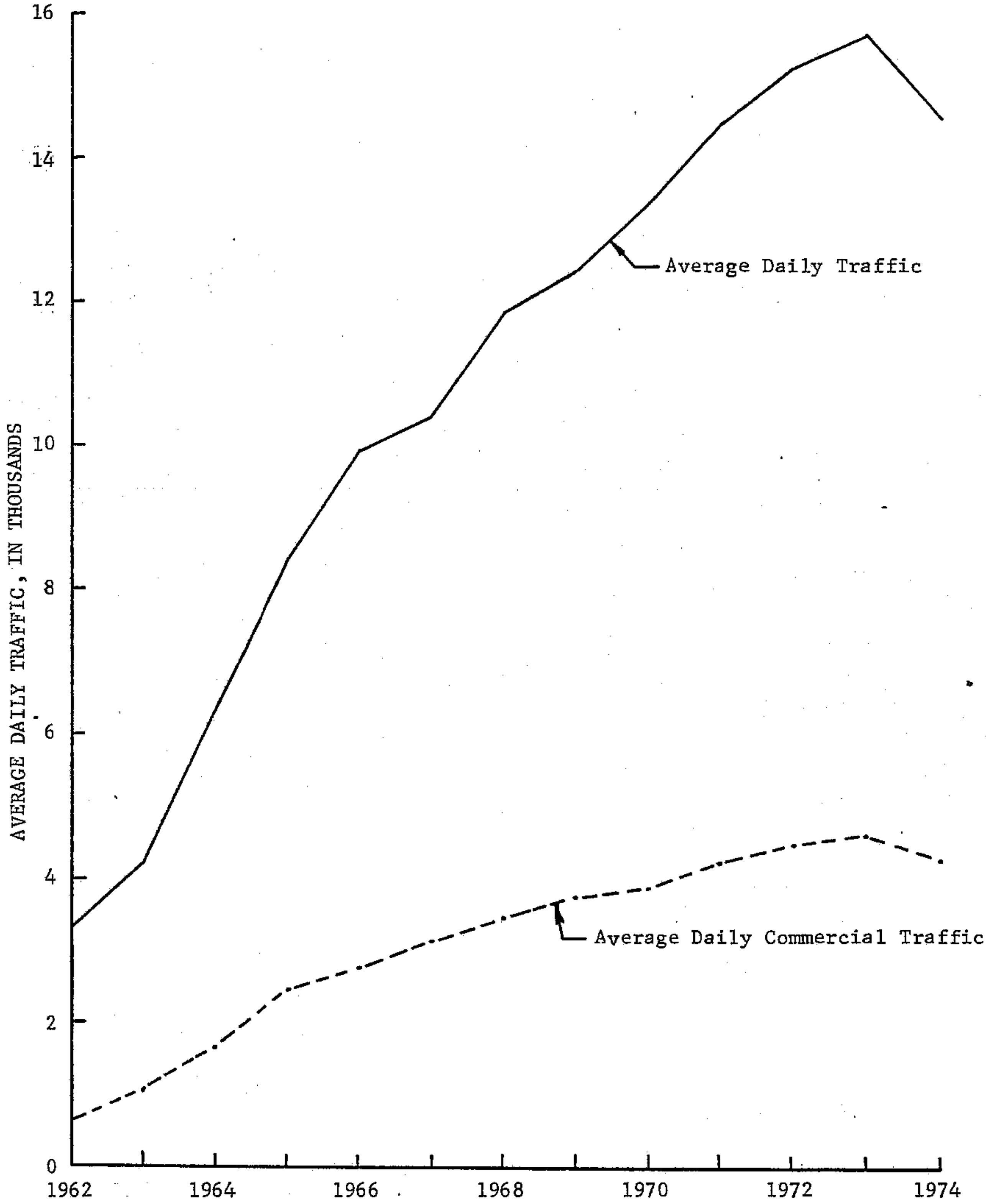


Figure 2. Average Daily Traffic and Average Daily Commercial Traffic from 1962 to 1974.

sections (Table 1) are tabulated by test loop, subbase type, slab thickness, and joint spacing. Each test section containing 15-ft joints was nonreinforced and 120 ft (36.58 m) long, while those containing 40-ft joints were reinforced and 240 ft (73.15 m) long. Joints at either end of a test section were excluded from the analysis because adjacent panel lengths differed and were not comparative.

The test sections built in 1962, which will be referred to as new test sections (Table 2), are arranged by subbase type, joint interval, and surface thickness. Those with 40-ft joints are 8-in. and 9.5-in. reinforced pavements mostly 240 ft (73.15 m) long, but a few are shorter. The test sections with a 100-ft joint interval are 10-in. reinforced pavements which range in length from 100 ft (30.48 m) to 3300 ft (1005.84 m). All the new test sections were built on an embankment of A-6 soil salvaged from the loop turnarounds. The new embankment is 3 ft (0.91 m) thick throughout except where a reduction in thickness was required to accommodate new thicker pavement designs and to provide the necessary clearance under the overhead structures.

The test sections listed in Table 3 are in the rigid pavement tangent of test Loop 1. Since Loop 1 has never been used for traffic, the behavior of contraction joints is associated only with environmental factors.

The AASHO test sections in the rehabilitated pavements (Table 1) have retained their originally assigned section numbers. New test sections are listed in Table 2. A zero, as the first digit in a section number, identifies a new section that is a replicate of an AASHO test section or a new section that is placed on a new type of subbase material other than the original sand-gravel material. Letters that appear in a section number identify new sections that were built as connecting links between the original test tangents. The 10-in. pavement sections that replaced AASHO test sections retained the original test section number. All test sections listed in

TABLE 1. ORIGINAL AASHO TEST SECTION GROUPS BY PAVEMENT DESIGN

Subbase Type	Test Loop	15-ft Joint (Nonreinforced) 8.0-in. Slab	9.5-in. Slab	11.0-in. Slab	12.5-in. Slab	40-ft Joint (Reinforced) 8.0-in. Slab	9.5-in. Slab	11.0-in. Slab	12.5-in. Slab
None	5		552						
SGM	4	652	676			692	646		
		658	690			696	666		
		672	702			670	668		
SGM	5		512	498			500	496	
			528	510			504	516	
			542	530			544	546	
							554		
SGM	6		352	364	350		382	338	356
			368	366	380		404	344	358
			376	378	396			346	360
			390	388				392	
				398					

Note: 1 in. = 254.0 mm; 1 ft = 0.3048 m

TABLE 2. TEST SECTION NUMBERS FOR NEW RIGID PAVEMENT DESIGNS

Subbase Type	40-ft Joints (Reinforced) 8.0-in. Slab	40-ft Joints (Reinforced) 9.5-in. Slab	100-ft Joints (Reinforced) 10.0-in. Slab
Bituminous Aggregate Material - (BAM)	070 082	072 080	058
Cement Aggregate Material - (CAM)	084 094	086 096	060
Sand-Gravel Material - (SGM)			340 348 386 506 508 550 D4C 2/
New SGM 1/	092	078	
Gravel	068 074	066 076	A2D A4D 342 384 D2C 490 520 538 548 C2B
Crushed Stone	064 088	062 090	

Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m

1/ New sand-gravel mixture that was obtained from an unused stockpile in a pit at the west end of the construction site.

2/ Old pilot test section.

TABLE 3. LOOP 1 TEST SECTION GROUPS BY PAVEMENT DESIGN

Surface Thickness (in.)	Joint 1/ Spacing (ft)	2/ Subbase Thickness 0	2/ Subbase Thickness 6-in.
2.5	40	895,897	899,931
	15	935	933
5.0	40	905	927
	15	899,893 903,923	891,901 925,929
9.5	40	907,921	887,915
	15	919	917
12.5	40	883	911
	15	881,885	909,913

Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m

1/ 15-ft joints nonreinforced
40-ft joints reinforced

2/ Sand-Gravel Material

the tables are in the outer traffic lane because more than 96 percent of the vehicles used that lane.

As an aid in locating test sections in the new roadway, Figures A-1 and A-2 have been included to show plan and elevation views of each test section.

Pavement Joints

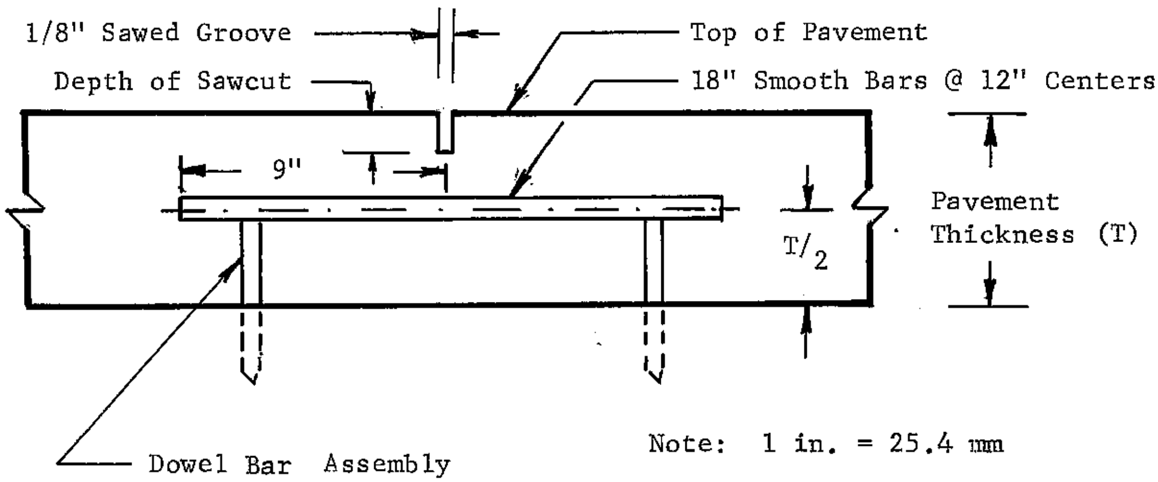
As stated previously, all the experimental rigid pavement sections, both the AASHO sections and the new sections, had sawed, dowelled, transverse, contraction joints. The dowel bars were smooth, round, carbon steel meeting the requirements of AASHO M 227 grade 70-80. They were supported by a wire assembly with sand plates which were secured to the subgrade or subbase by pins. The dowel bar assemblies used in the AASHO pavement were coated with two applications of coal-tar base mill coating applied at the fabrication plant, and the bars used in the new pavement had one application of heavy cup grease applied just prior to paving.

The AASHO pavement was sawed during the morning following placement of the concrete. The saw, which operated on the forms, had four tandem diamond blades After sawing, the joints were cleaned and were filled with jute roving. When construction was completed, the jute roving was removed, the joints were cleaned and were sealed with a cold-applied sealant conforming to ASTM D 1850.

The new pavement was sawed with a single-blade machine the day following placement. Then, the sawed joint was cleaned and was sealed with the same cold-applied asphaltic joint sealant used in the original construction.

Sawcut depth, dowel bar diameter, and pavement fabric wire gage varied with the slab thickness. Sawed contraction joint details including dimensions can be seen in Figure 3.

After the initial sealing, nothing further in the form of maintenance was done until 1965. At that time all joints and cracks were resealed with a PAF-2 modified, hot-poured joint sealant. No further resealing was done in either joints or cracks.



Pavement Thickness (T) (in.)	Depth of Sawcut (in.)	Dowel Bar Diameter (in.)	Pavement Fabric 1/ Style	Pavement Fabric Depth (in.)
8	1-3/4	1	612-33	2
9-1/2	2	1-1/4	612-22	2
10	2-3/4	1	612-004	2-1/2
11	2-1/4	1-3/8	612-11	2
12-1/2	2-1/2	1-5/8	612-00	2

1/ Fabric Style Code 6 1 2 - 2 2
- 2 (second) — transverse wire gage
- 2 (first) — longitudinal wire gage
- 2 — transverse wire spacing
- 6 — longitudinal wire spacing

Figure 3. Detail of sawed contraction joint and dimensions of pavement accessories.

Analysis Procedure

The experimental pavement sections, which have yielded data for the study of transverse joint behavior, represent two separate experiments. The first involves all the pavement sections that were part of the original Road Test experiment. They are listed in Table 1 where test sections are arranged by joint interval, by pavement slab thickness, by subbase type and by original test loop. The subdivision into test loop categories separates the test sections into groups that have carried the same number of 18-kip equivalent single-axle load applications.

The second involves all the test sections that were added to the test facility in 1962. They are listed in Table 2. These sections are arranged by joint interval, by pavement slab thickness and by subbase type. Since this experiment postdates the AASHO Road Test, the test sections have received axle-load applications from only regular highway traffic. The 10-in. pavement sections in this experiment vary in length from 100 ft (30.48 m) to 3300 ft (1005.84 m). The 8.0-in. and 9.5-in. test sections are either 120 ft (36.58 m) or 240 ft (73.15 m) long. The test section groups in Table 3 are those from the original rigid pavement tangent in Loop 1. All sections in Loop 1 have been affected only by environment since axle loads have never been applied.

Once the test section groupings had been established, the data could be combined in various ways to make the comparisons that are described in the following sections of this report. Only simple averages have been used because the shortage of test data in some design categories precludes the rigorous application of statistical techniques. In studying joint behavior, the joints that marked the beginning and the end of a test section in Loops 4, 5, and 6 were excluded from analysis because the adjacent panels in transitions differed either in panel length or in cross-section, but those in Loop 1 were retained because a number of Loop 1 test sections contained only one panel. In studying transverse cracking and pavement smoothness, no exceptions were made. Joint behavior, transverse cracking, pavement smoothness and Loop 1 deterioration are described in the following sections of the report.

JOINT BEHAVIOR

In this section the behavior of dowelled, transverse contraction joints in pavements having the same joint interval, pavement thickness and subbase type is compared with that of similar pavements having a different joint interval. Joint behavior is examined also while holding joint interval constant but varying pavement thickness, varying subbase type, and varying axle loading. Behavior is expressed as the average amounts of spalling, faulting, joint opening, compression cracking and D-cracking which occurred at joints. Pumping, corner breaking and pavement buckling were insufficient to be considered as variables in joint behavior. Minor pumping was observed after heavy rains in several sections, particularly near the east end of the test facility. No corner breaks have been observed in any test section. To date, pavement buckling (blowup) has occurred only twice, once in 1963 (Section 895 Loop 1) and again in 1969 (original pilot test section next to the electronic scale pit).

Spalling

Spalling occurs when incompressible materials, which infiltrate a joint, resist joint closure during warm weather and produce shear stress points that exceed the shear strength of the concrete. When this occurs, part of the joint edge breaks away. In the field survey, spalling was classified into four categories depending on the width of the spalls. Class 1 spalls are less than 5/8 in.

(16 mm) wide, and Class 2 ranges from 3/8 in. (10 mm) up to 1 3/4 in. (44 mm) wide. Class 3 spalls range from 1 3/4 in. (44 mm) up to 3 in. (76 mm) and Class 4 exceeds 3 in. (76 mm) in width. For this report, Class 1 and Class 2 were combined as minor spalls, and Class 3 and Class 4 were combined as major spalls.

The mean number of minor spalls and of major spalls per pavement mile is shown in Table 4. The table is arranged by joint interval, by slab thickness, by subbase type, and by load applications. In general, the mean number of both minor and major spalls per pavement mile increased as the number of joints increased, but the amount of spall per joint was not clearly related to joint interval. The number of major spalls (Table 4) would have been larger except that some were filled with bituminous material and have been counted as patches in the Patching Summary, Table 6.

Faulting

Faulting, which is identified by the drop developed from the approach slab to the leave slab, is a type of joint deterioration produced by traffic. When water collects under a pavement, the downward thrust suddenly applied to the leave slab, as a wheel load crosses a joint, forces loose wet material back under the approach slab. This scouring action eventually raises the elevation of the approach slab. Faulting was noticed first in 1966 when a few faults were observed in Loop 4. Since that time, faulting has progressed continuously. In February 1973, the joints in every test section were measured with the fault gauge shown in Figure 4. The fault measurements (Figures 5 and 6) indicate that pavement thickness, subbase type, and joint spacing affected the amounts of faulting that occurred.

Cumulative frequency curves of faulting in the AASHO test sections are shown in Figure 5. The cumulative frequency of faulting for each pavement thickness

TABLE 4. MEAN NUMBER OF SPALLS PER PAVEMENT MILE 1/

Joint Interval (ft)	Joints Per Mile	Surface Thickness (in.)	18-KIP ESAL (millions)	None		Granular		Stabilized	
				Minor	Major	Minor	Major	Minor	Major
				NONREINFORCED					
15	352	8	11.6	-	-	220	59	-	-
		9.5	11.6	-	-	220	235	-	-
		9.5	14.1	704	0	147	0	-	-
		9.5	18.6	-	-	165	35	-	-
		11.0	14.1	-	-	176	0	-	-
		11.0	18.6	-	-	282	9	-	-
		12.5	18.6	-	-	220	0	-	-
				REINFORCED					
40	132	8.0	10.0	-	-	84	21	57	57
		8.0	11.6	-	-	59	37	-	-
		9.5	10.0	-	-	55	14	88	22
		9.5	11.6	-	-	88	59	-	-
		9.5	14.1	-	-	94	6	-	-
		9.5	18.6	-	-	110	0	-	-
		11.0	14.1	-	-	51	0	-	-
		11.0	18.6	-	-	105	11	-	-
		12.5	18.6	-	-	15	0	-	-
100	52.8	10.0	10.0	-	-	24	12	39	13

Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m; 1 mile = 1.609 km; 1 kip = 453.6 kg

1/ (a) Minor spalls are less than 1.25-in. wide. Include Class 1 & 2 spalls.
(b) Major spalls are more than 1.25-in. wide. Include Class 3 & 4 spalls.

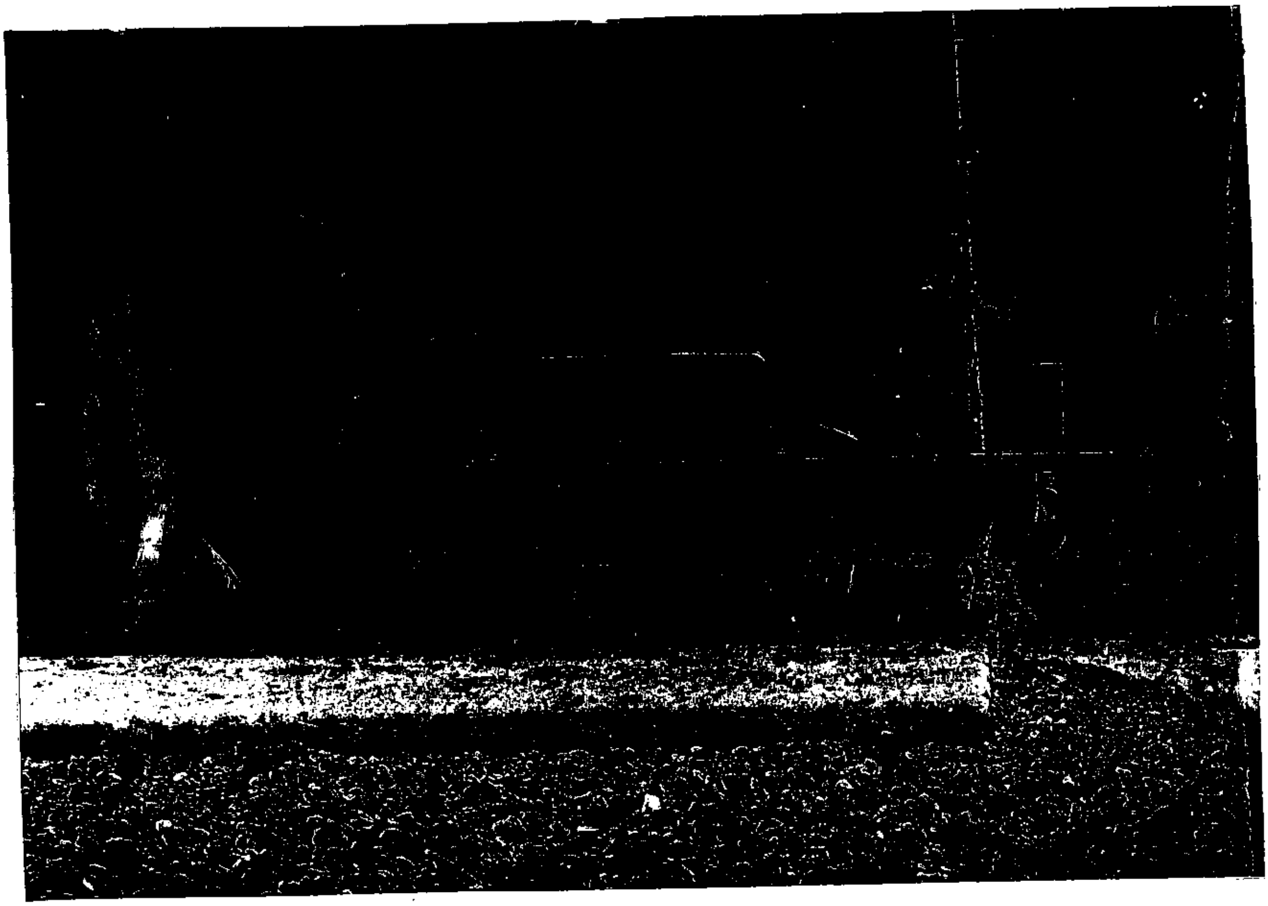
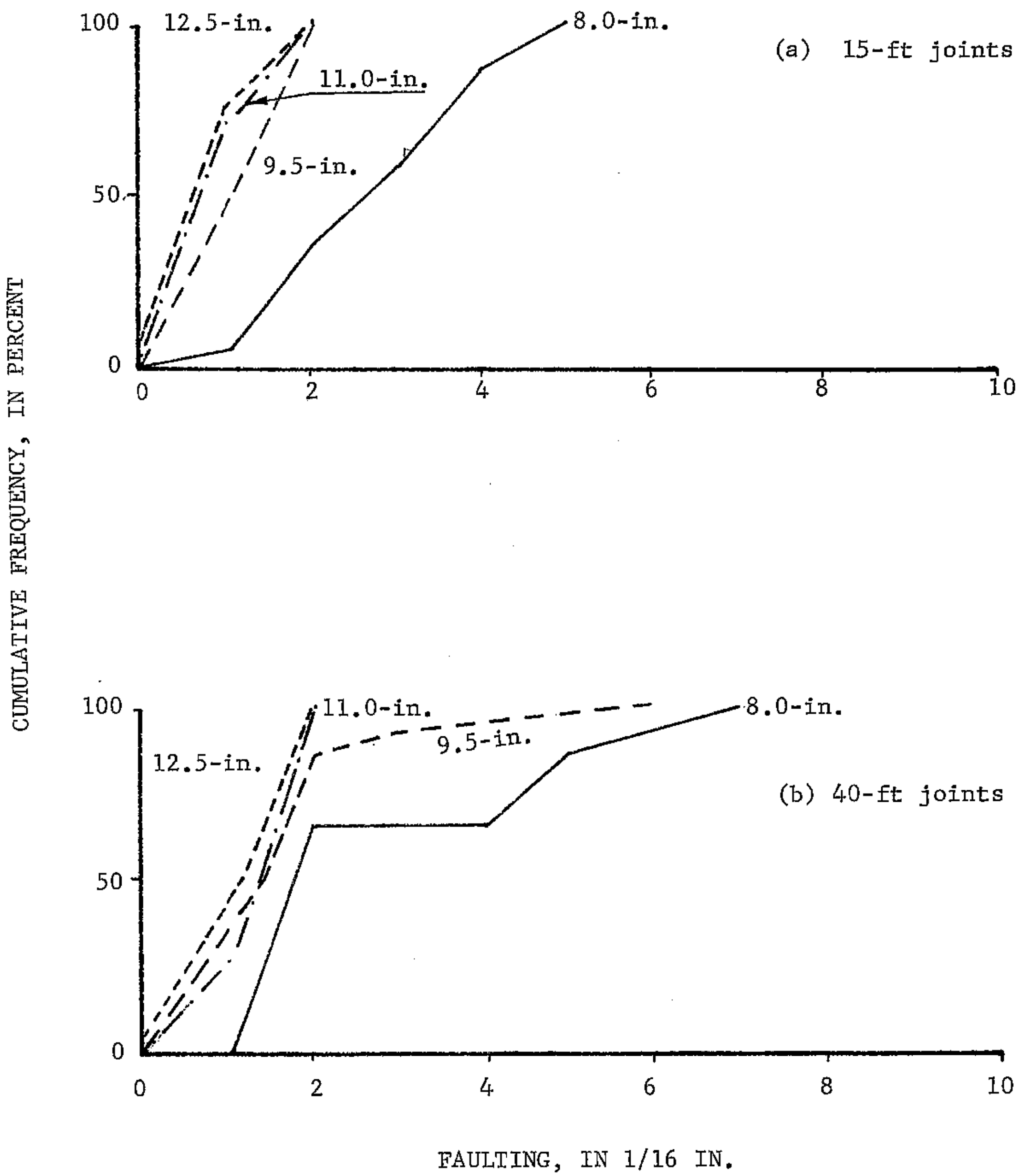
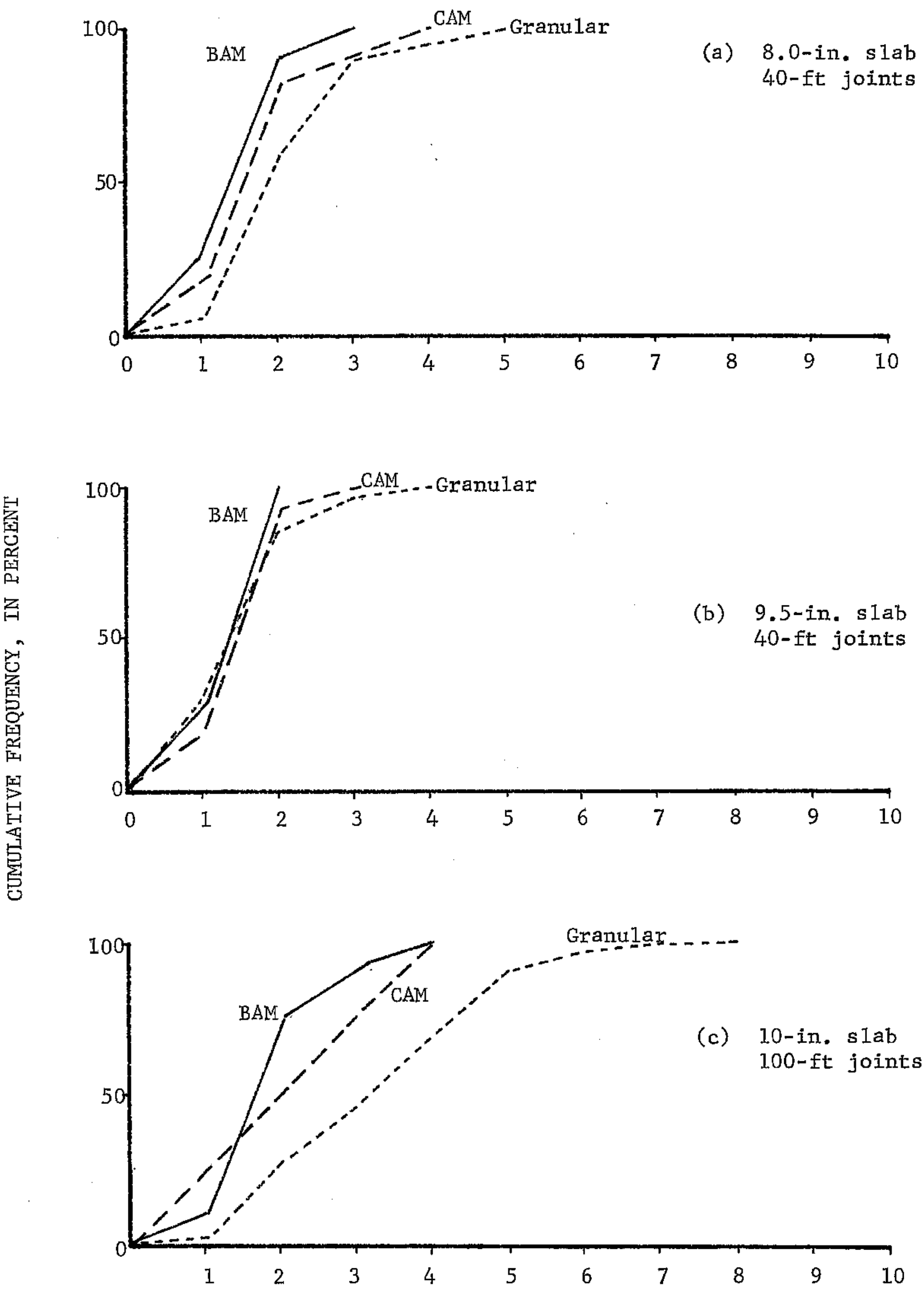


Figure 4. Photograph of fault gage.



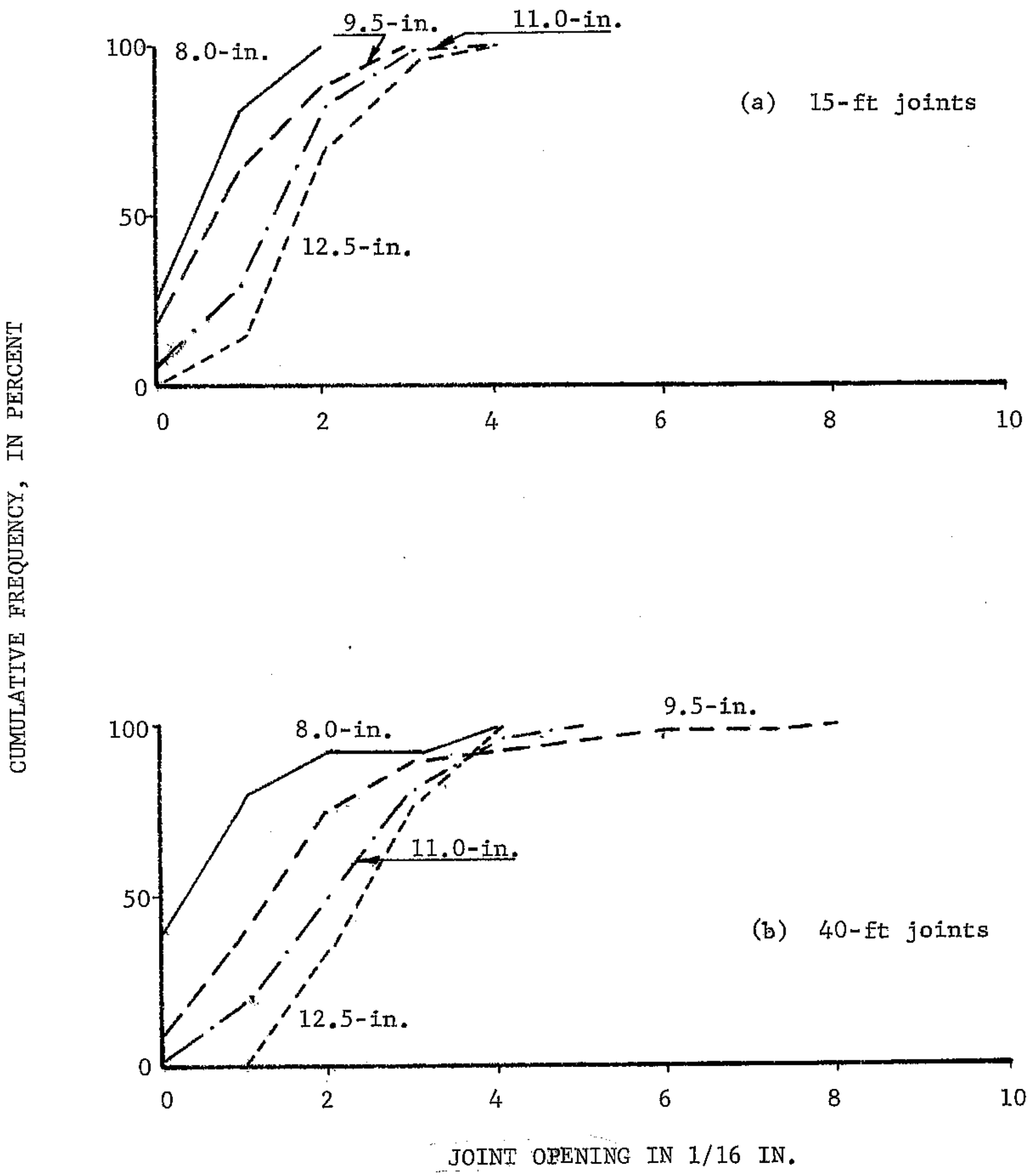
Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m

Figure 5. Cumulative frequency of faulting in original AASHO pavements according to pavement thickness.



Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m

Figure 6. Cumulative frequency of faulting in new pavements by subbase type.



Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m

Figure 7. Cumulative frequency of winter joint opening in original AASHO pavement by thickness.

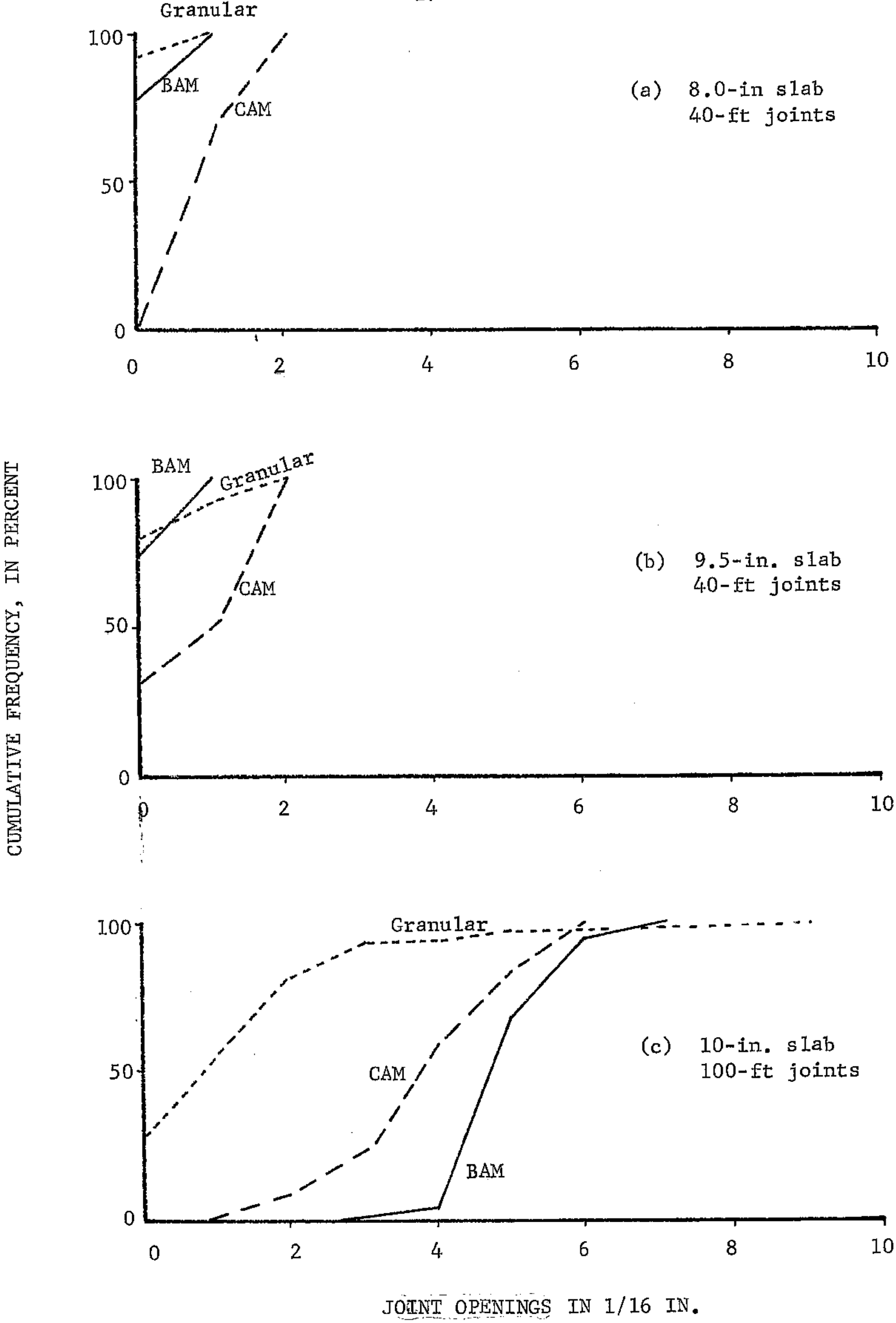
The frequency curves in Figure 7a suggest that, for the climatic conditions in the Ottawa area, joint spacings less than 15 ft (4.57 m) would be required to provide dependable load transfer through aggregate interlock. According to NCHRP Synthesis of Highway Practice No. 19 (11), aggregate interlock is lost when joint openings exceed 1/32 in. (0.8 mm). As can be seen in the figure, a minimum of 50 percent of the joints were opened in excess of this amount. Therefore, aggregate interlock alone would not have been a dependable load-transfer method in the test pavements.

The frequency curves of winter joint openings in the new test sections are shown in Figure 8 and are arranged by subbase type. In Figure 8c it can be seen that most of the pavement joints in the 100-ft joint spacing on the stabilized subbase have opened at least 1/8 in. (3.2 mm). In fact, 92 percent of the joints in the pavement on the BAM subbase and 45 percent of the joints on the CAM subbase were opened from 1/4 in. (6.4 mm) to 7/16 in. (11.1 mm). On the other hand, 28 percent of the joints on the granular subbase were closed, while the open joints ranged from 1/16 in. (1.6 mm) to 9/16 in. (14.3 mm) in width. Moreover, joint openings for the 100-ft panels were more uniform on the stabilized subbases than on the granular subbase.

As can be seen in Figures 8a and 8b, there were so many inoperative joints in the new test sections with the 40-ft joint interval that reliable conclusions could not be reached relative to the effect of subbase type on winter joint openings.

D-Cracking

D-cracking is a form of pavement disintegration that is a series of closely spaced fine cracks that become visible in the pavement wearing surface adjacent to and sub-parallel to transverse and longitudinal joints and cracks, and to exposed



Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m

Figure 8. Cumulative frequency of winter joint openings for new pavements by subbase type.

pavement edges. Usually, these cracks contain a grayish-blue deposit of calcareous material. Laboratory and field research by others indicates that D-cracking is caused by stresses generated during the freezing of critically saturated coarse aggregate particles (11). The cracks widen, allowing pavement fragments to break away. Usually, pavement disintegration first appears at the intersection of transverse and longitudinal joints and at the outside corner of a pavement slab. As this form of pavement distress progresses it can contribute significantly to a loss in serviceability, especially during the final stage of pavement service life.

The area of D-cracking in square feet per pavement mile and in square feet per joint in both the original and the new test sections is listed in Table 5. The table is arranged according to joint interval, surface thickness, axle load applications and subbase type. As can be seen in the table, the largest amount of D-cracking occurred in the test sections that had 14.1 million 18-kip ESAL'S (Loop 5). Lesser amounts of D-cracking occurred in Loop 6 (18.6 18-kip ESAL'S) and in Loop 4 (11.6 18-kip ESAL'S), respectively. Variations among test loops is unexplained by the parameters investigated.

Other than the loop effect, the table indicates that the total amount of D-cracking was larger in the older pavements than in the newer pavements. However, D-cracking per pavement mile was largest in the sections with the most joints although the amount of damage per joint was largest in the pavements with the fewest joints. No differences in D-cracking could be associated with the type of subbase.

Patching

Patches consist of small areas of bituminous cold mix placed in damaged areas by maintenance crews. The patched areas (skin patches) most often occurred at the pavement joints where the damage resulted from spalling and D-cracking. Patching

TABLE 5. MEAN AREA OF D-CRACKING IN SQ. FT. PER MILE OF PAVEMENT

Joint Interval (ft)	Surface Thickness (inches)	Applications 18-K ESAL'S (millions)	SUBBASE TYPE: None		Granular		Stabilized	
			Per Mile	Per Joint	Per Mile	Per Joint	Per Mile	Per Joint
			NONREINFORCED					
15	8.0	11.6	-	-	29	0.1	-	-
	9.5	11.6	-	-	59	0.2	-	-
	9.5	14.1	924	2.6	1159	3.3	-	-
	9.5	18.6	-	-	220	0.6		
	11.0	14.1	-	-	601	1.7	-	-
	11.0	18.6	-	-	185	0.5	-	-
	12.5	18.6	-	-	51	0.1	-	-
			REINFORCED					
40	8.0	10.0	-	-	0	0	34	0.3
	8.0	11.6	-	-	15	0.1	-	-
	9.5	10.0	-	-	109	0.8	0	0
	9.5	11.6	-	-	37	0.3	-	-
	9.5	14.1	-	-	413	3.1	-	-
	9.5	18.6	-	-	264	2.0	-	-
	11.0	14.1	-	-	352	2.7	-	-
	11.0	18.6	-	-	314	2.4	-	-
	12.5	18.6	-	-	249	1.9	-	-
100	10.0	10.0	-	-	153	2.9	161	3.1

Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m; 1 mile = 1.609 km; 1 kip = 453.6 kg.

first appeared in 1966, and at that time the amounts of patching in AASHO test sections and in new test sections were about equal. Seven percent of the AASHO test sections and 8 percent of the new test sections had received some patching. The largest patched areas amount to approximately 1 sq ft (0.09 m^2). The number of patches and the size of patches increased as time passed. By 1974 85 percent of the AASHO sections and 38 percent of the new sections had at least one patch.

The mean areas of patching at the joints in 1974 are in Table 6. In general, the amount of patching at each joint decreased as pavement thickness increased. Patching also decreased as joint interval decreased, but the cumulative amount of patching per pavement mile increased as the joint interval decreased. The AASHO test sections had more patching per pavement mile than the new test sections, and the test sections on a stabilized subbase had slightly more patching than those on a granular subbase.

Compression Cracks

Compression cracks are tight cracks that extend perpendicularly from the joint for several inches and then commonly divide into two or more branches. Compression cracks also have been referred to as "stringers" and as "crowsfeet" in the literature. They usually form in the outer wheelpath, but occasionally they form in the inner wheelpath. The number of compression cracks per mile of pavement is tabulated in Table 7 by subbase type, by pavement thickness, and by joint interval for both the new and the original test sections. The amount of compression cracking that developed was so small that neither a loss in pavement serviceability nor a need for maintenance resulted.

PAVEMENT BEHAVIOR

In this section the behavior of the pavements in the rehabilitated test road with respect to transverse cracking and to pavement roughness is presented. The environomental effects observed in Loop 1 are included also.

TABLE 6. MEAN AREA (SQUARE FEET) OF PATCHING BY PAVEMENT MILE AND BY JOINT

Joint Interval (ft)	Joints Per Mile	Surface Thickness (in.)	18-KIP ESAL (millions)	SUBBASE TYPE					
				None		Granular		Stabilized	
				Per Mile	Per Joint	Per Mile	Per Joint	Per Mile	Per Joint
				NONREINFORCED					
15	352	8	11.6	-	-	572	1.6	-	-
		9.5	11.6	-	-	132	0.4	-	-
		9.5	14.1	572	1.6	249	0.7	-	-
		9.5	18.6	-	-	110	0.3	-	-
		11.0	14.1	-	-	176	0.5	-	-
		11.0	18.6	-	-	88	0.3	-	-
		12.5	18.6	-	-	59	0.2	-	-
				REINFORCED					
40	132	8.0	10.0	-	-	53	0.4	52	0.4
		8.0	11.6	-	-	264	2.0	-	-
		9.5	10.0	-	-	5	0.03	138	1.0
		9.5	11.6	-	-	242	1.8	-	-
		9.5	14.1	-	-	50	0.4	-	-
		9.5	18.6	-	-	44	0.3	-	-
		11.0	14.1	-	-	110	0.8	-	-
		11.0	18.6	-	-	50	0.4	-	-
		12.5	18.6	-	-	66	0.5	-	-
100	52.8	10.0	10.0	-	-	14	0.3	20	0.4

Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m; 1 mile = 1.609 km; 1 kip = 453.6 kg.

TABLE 7. MEAN NUMBER OF COMPRESSION CRACKS PER MILE OF PAVEMENT

Joint Spacing	Slab Thickness (inches)	Subbase Type: None	Subbase Type: Granular	Subbase Type: Stabilized
			Original AASHO Sections	
15-ft	8.0	0	0 1/	
	9.5		5(.01)	
	11.0		12(.03)	
	12.5		67(.20)	
40-ft	8.0		9(.07)	
	9.5		12(.09)	
	11.0		8(.06)	
	12.5		53(.40)	
			New Sections	
40-ft	8.0		16(.10)	7(.05)
	9.5		6(.05)	14(.10)
100-ft	10.0		16(.30)	8(.15)

1/ Number enclosed by parentheses is the average number of compression cracks per joint.

Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m; 1 mile = 1.609 km

Transverse Cracking

Controlling random transverse cracking, which commonly develops in conventional pavements, has concerned engineers ever since portland cement concrete has been used for highway pavements. Over the years, Illinois, like many other states, has continually experimented with a variety of transverse joints and transverse joint intervals (2, 12). As previously mentioned, sawed dowelled contraction joints spaced at 100 ft (30.48 m) were the Illinois standard from 1955 through 1975. Experience indicates that from two to eight cracks per 100-ft panel will occur in conventional 10-in. pavements. The majority of these cracks are prevented from opening by steel pavement fabric, but when the steel breaks as it does in a number of instances, the crack can widen, and openings of 1/4 in. (6 mm) or more are common. The experimental rigid pavement sections in the rehabilitated test road provide an opportunity to compare cracking behavior in pavements with joints spaced at 15 ft (4.57 m), 40 ft (12.19 m), and 100 ft (30.48 m).

Transverse cracks are defined as all cracks that have the longest projected length perpendicular to the pavement centerline. They have been divided into four classes: Class 1, which includes fine cracks not discernible under dry conditions beyond 15 ft (4.57 m); Class 2, which includes cracks that are discernible at 15 ft (4.57 m) but not more than 1/4 in. (6 mm) wide at the surface; Class 3, which includes cracks that are open or spalled at the surface to a width of 1/4 in. (6 mm) or more over one-half of their length; and Class 4, which includes all sealed cracks. Major cracks comprise Class 3 and Class 4 cracks. The mean number of total cracks per mile of pavement and the mean number of major cracks per mile for each slab thickness, subbase type and joint interval are listed in Table 8.

As seen in the table, the number of transverse cracks increases as load applications increase and as slab thickness decreases. Although unexpected, a few

TABLE 8. MEAN NUMBER OF TRANSVERSE CRACKS PER PAVEMENT MILE

Joint Interval (ft)	Surface Thickness (inches)	Applications 18-K, ESAL'S (millions)	SUBBASE TYPE: None		Granular		CAM		BAM	
			Total	Major	Total	Major	Total	Major	Total	Major
			NONREINFORCED							
15	8.0	11.6	-	-	103	103	-	-	-	-
	9.5	11.6	-	-	0	0	-	-	-	-
	9.5	14.1	0	0	15	15	-	-	-	-
	9.5	18.6	-	-	44	22	-	-	-	-
	11.0	14.1	-	-	0	0	-	-	-	-
	11.0	18.6	-	-	0	0	-	-	-	-
	12.5	18.6	-	-	0	0	-	-	-	-
			REINFORCED							
40	8.0	10.0	-	-	285	90	165	33	168	0
	8.0	11.6	-	-	403	212	-	-	-	-
	9.5	10.0	-	-	182	118	209	66	132	44
	9.5	11.6	-	-	264	154	-	-	-	-
	9.5	14.1	-	-	330	143	-	-	-	-
	9.5	18.6	-	-	319	110	-	-	-	-
	11.0	14.1	-	-	286	66	-	-	-	-
	11.0	18.6	-	-	275	121	-	-	-	-
	12.5	18.6	-	-	235	37	-	-	-	-
100	10.0	10.0	-	-	339	34	205	10	113	0

Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m; 1 kip = 453.6 kg

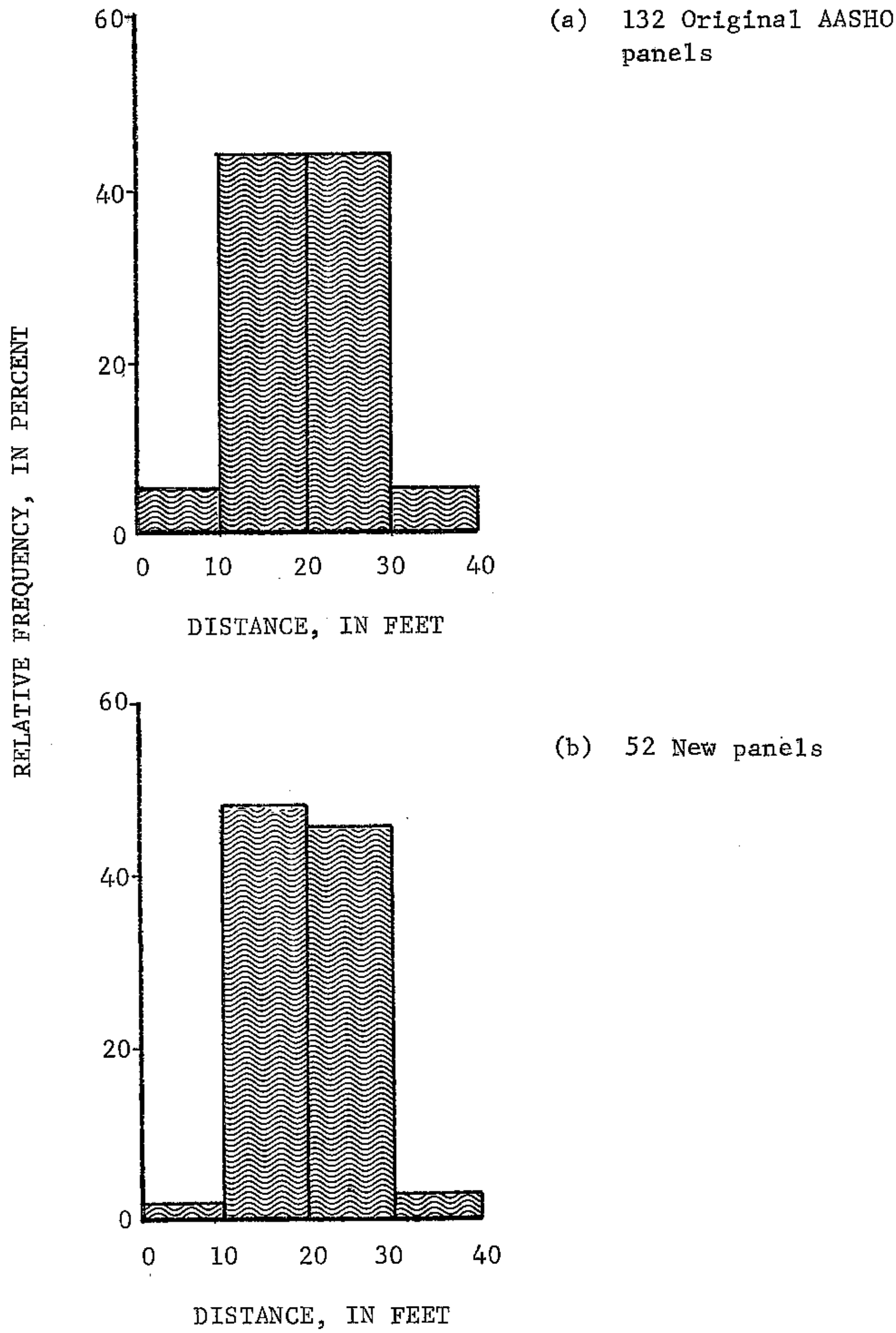
Major cracks are defined as Class 3 and Class 4 cracks

cracks did form in the 8-in. and 9.5-in. nonreinforced 15-ft pavement panels. In the reinforced 40-ft panels, the AASHO pavement averaged 2.7 cracks per panel as compared to 1.4 cracks per panel in the new pavement. The new pavement with 100-ft joints averaged 4.1 cracks per panel.

The new test sections were placed on a granular, a CAM, and a BAM subbase. On the average, more transverse cracks formed in the pavements placed on a granular subbase than in the pavement placed on a stabilized subbase. On the granular subbase there were 1.7 cracks per 40-ft panel and 6.4 cracks per 100-ft panel as compared to 1.4 cracks per 40-ft panel and 3.9 cracks per 100-ft panel on the CAM subbase, and only 1.1 cracks per 40-ft panel and 2.1 cracks per 100-ft panel on the BAM subbase. On the average, all stabilized subbases had 1.3 cracks per 40-ft panel and 3.0 cracks per 100-ft panel.

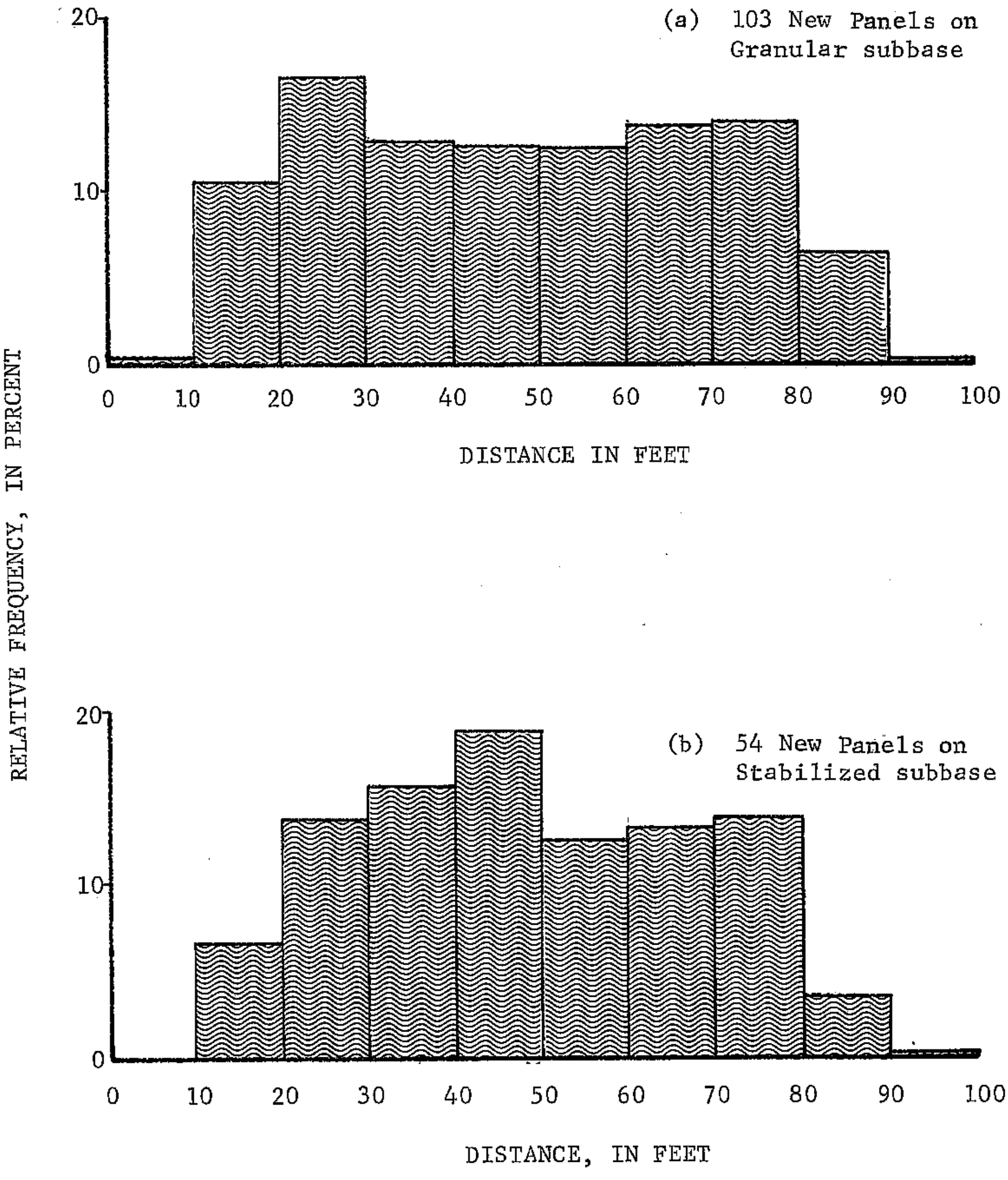
The number of major cracks per pavement mile on the various subbases also was related to the number of inoperative joints in a pavement panel. As can be seen in Figure 8 and in Table 8, the greatest number of major cracks occurred in the pavement on the granular subbase where the greatest number of inoperative joints were found. Conversely, the pavement sections on the stabilized subbases, where the least number of inoperative joints were found, had relatively few major cracks. The number of major cracks was least for the BAM subbase sections and intermediate for the CAM sections. Thus, the tensile stresses, which caused cracks to form in the pavement panels must be lower on the stabilized subbase, particularly BAM, than on the granular subbase.

To gain further insight into transverse cracking in the pavement panels, relative frequency histograms were prepared. The frequency histograms for the 40-ft panels on a granular subbase are in Figure 9 while those for the 100-ft panels



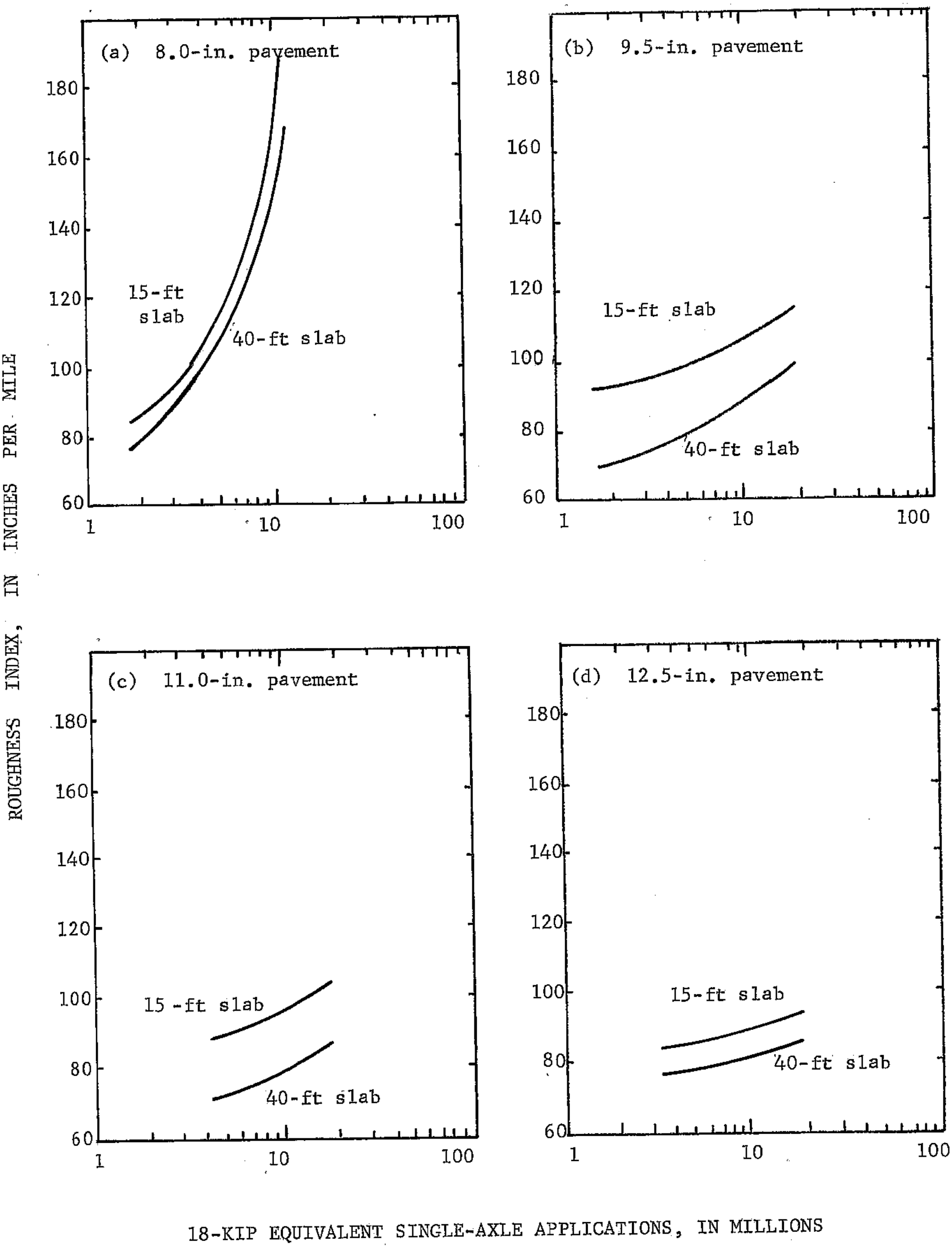
Note: 1 ft = 0.3048 m

Figure 9. Relative frequency distribution of transverse cracks in 40-ft panels overlying a granular subbase.



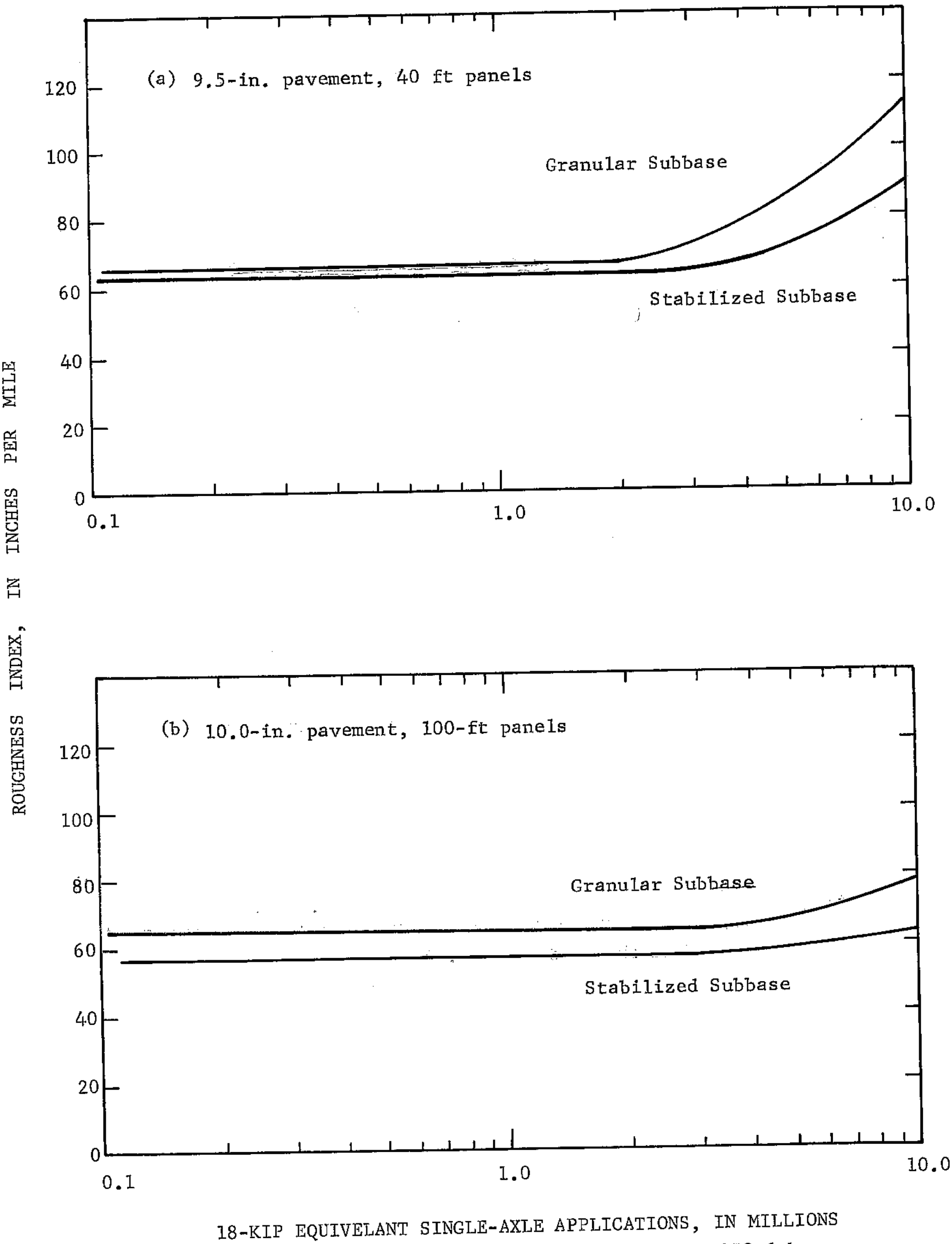
Note: 1 ft = 0.3048 m

Figure 10. Relative frequency distribution of transverse cracks in new 100-ft slabs.



Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m; 1 Kip = 453.6 kg

Figure 11. Change in Roughness Index with traffic, AASHO sections.



Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m; 1 Kip = 453.6 kg

Figure 12. Change in Roughness Index with traffic, new sections.

and is believed to be the direct result of the difference in the total number of transverse joints per unit length of pavement. Although the 8-in. pavements with the 40-ft joints were smoother than those with the 15-ft joints (Figure 11a), the difference in smoothness is associated with the number of joints per unit pavement length and not with the presence or absence of pavement reinforcement. This is in agreement with the findings of the AASHO Road Test. The abrupt rise in pavement roughness of the 8-in. pavement sections shows that they were near the end of their service life.

The change in pavement roughness with increasing numbers of load applications in the new pavement sections with joints spaced at 40 ft (12.19 m) and at 100 ft (30.48 m) is plotted in Figure 12. Pavement sections 9.5 in. (241 mm) thick on the granular subbase are compared to those on the stabilized subbase in Figure 12a and the pavement sections 10 in. (254 mm) thick on a granular subbase are compared to those on a stabilized subbase in Figure 12b. As can be seen in Figure 12, subbase type had little effect on roughness in the 9.5-in. pavement sections early in their service life, but later the sections on a granular subbase began to increase in roughness at a greater rate than those on the stabilized (BAM and CAM) subbases. In comparing the stabilized subbase to the granular subbase in both Figure 12a and Figure 12b, the increase in pavement roughness (RI) with axle-load applications was less for the pavements on a stabilized subbase.

Behavior Data, Loop 1

In addition to the four main test loops, which carried traffic during the AASHO Road Test, Loop 1 was constructed as a non-traffic loop where the pavement could be observed as affected only by environment. Test sections in this loop are 2.5-in., 5.0-in., 9.5-in., and 12.5-in. pavements either with or without a 6-in. granular subbase and with either 15-ft or 40-ft pavement panels. Most of the test sections consist of a single, unjointed pavement panel either 15 ft (4.57 m) or 40 ft (12.19 m) long with a transition zone at either end. This arrangement destroys uniformity between adjacent pavement panels across transverse joints. Nevertheless, the D-cracking, the spalling, and the transverse cracking, which have occurred in the pavement panels, have been summarized and are presented in Table 9. Even though Loop 1 has carried no traffic, D-cracking and spalling are noticeable at joints, particularly in pavements placed directly on the subgrade. Very little transverse cracking occurred.

JOINT EXAMINATION

In 1975, prior to the resurfacing of the rigid pavement sections, two construction joints were removed for examination. Because the examination was limited to only two joints, they were selected from the 9.5-in. pavement, which is close to the Illinois standard 10-in. pavement. One joint was selected from AASHO test section 382, which overlies a 3-in. granular subbase, and the other joint was selected from new test section 086, which overlies a 4-in. CAM subbase.

The joints were removed by making full-depth transverse saw cuts 2 ft (0.6 m) on each side of the joint and by making two longitudinal cuts across the joint, one at the centerline and another 6 ft (1.8 m) from the pavement edge. The slabs were removed from the pavement and were transported without disturbing the joints. One half of each joint was sent to the University of Illinois, who cooperated in the investigation by conducting the pullout test, while the other half of each joint was examined by the Division of Highways.

The following sections describe details observed during the examination, and discuss results of concrete tests and of joint pullout tests.

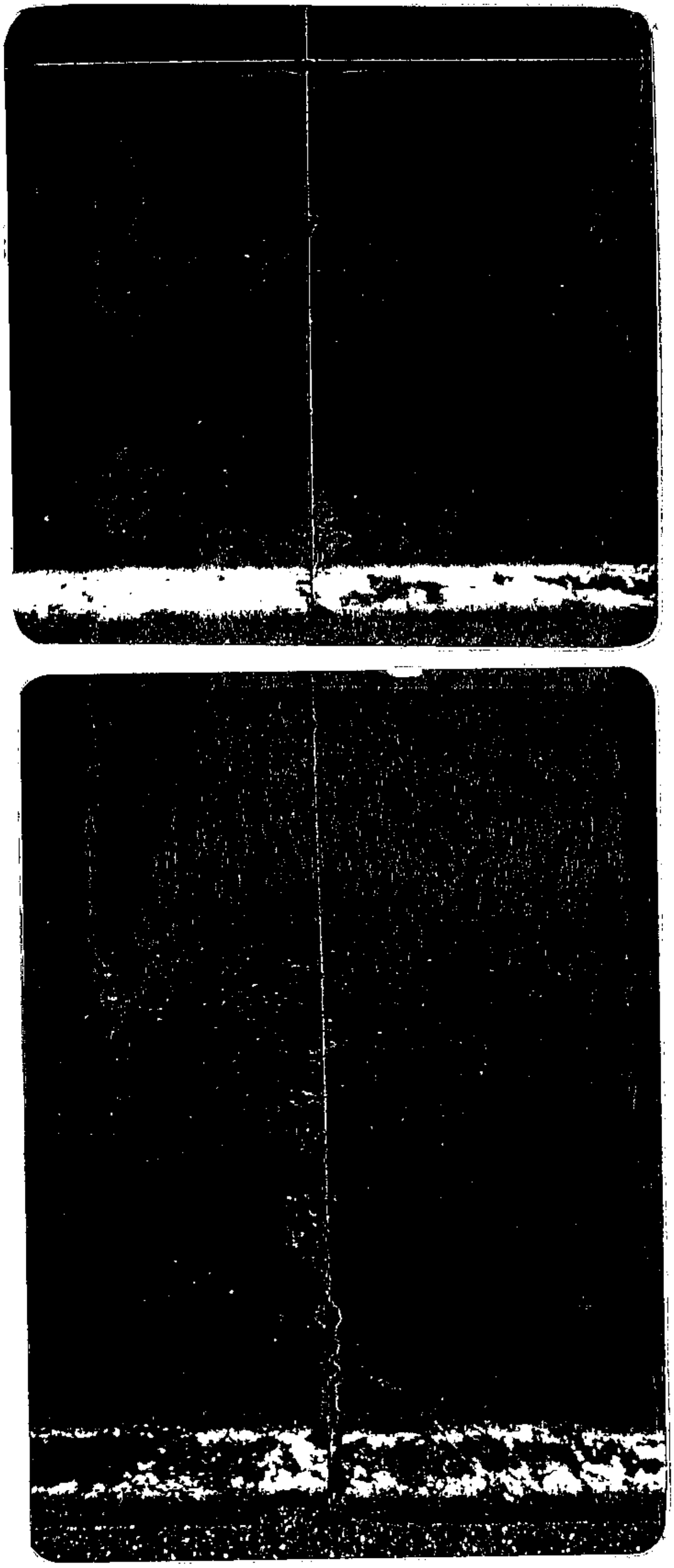
General Observations

The general appearance of the joints at the pavement surface can be seen in Figure 13, and their undersides can be seen in Figure 14. The photographs show

TABLE 9. BEHAVIOR OF PAVEMENT IN LOOP 1 (NO TRAFFIC)

Joint Spacing	Slab Thickness (inches)	D-Cracking (Sq ft Per Slab)		Spalling (Sq ft Per Slab)		Transverse Cracks (Per Slab)	
		Subbase Thickness (inches)					
		0	6	0	6	0	6
15-ft	2.5	0	0	0	0	1	0
	5.0	2.7	0.2	0.3	0.3	0	0
	9.5	0	3.0	2.0	0	0	0
	12.5	8.0	1.0	3.0	0	0	0
40-ft	2.5	0	0	0.5	0	0.5	1.0
	5.0	1.5	1.0	1.0	0	0	0
	9.5	2.5	1.5	0.5	0.5	0.5	0.5
	12.5	0	0	0	0	0	0

Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m; 1 sq ft = 0.0929 m^2



Section 382

Section 086

Figure 13. Photographs showing condition of joints removed for testing from Sections 382 and 086.

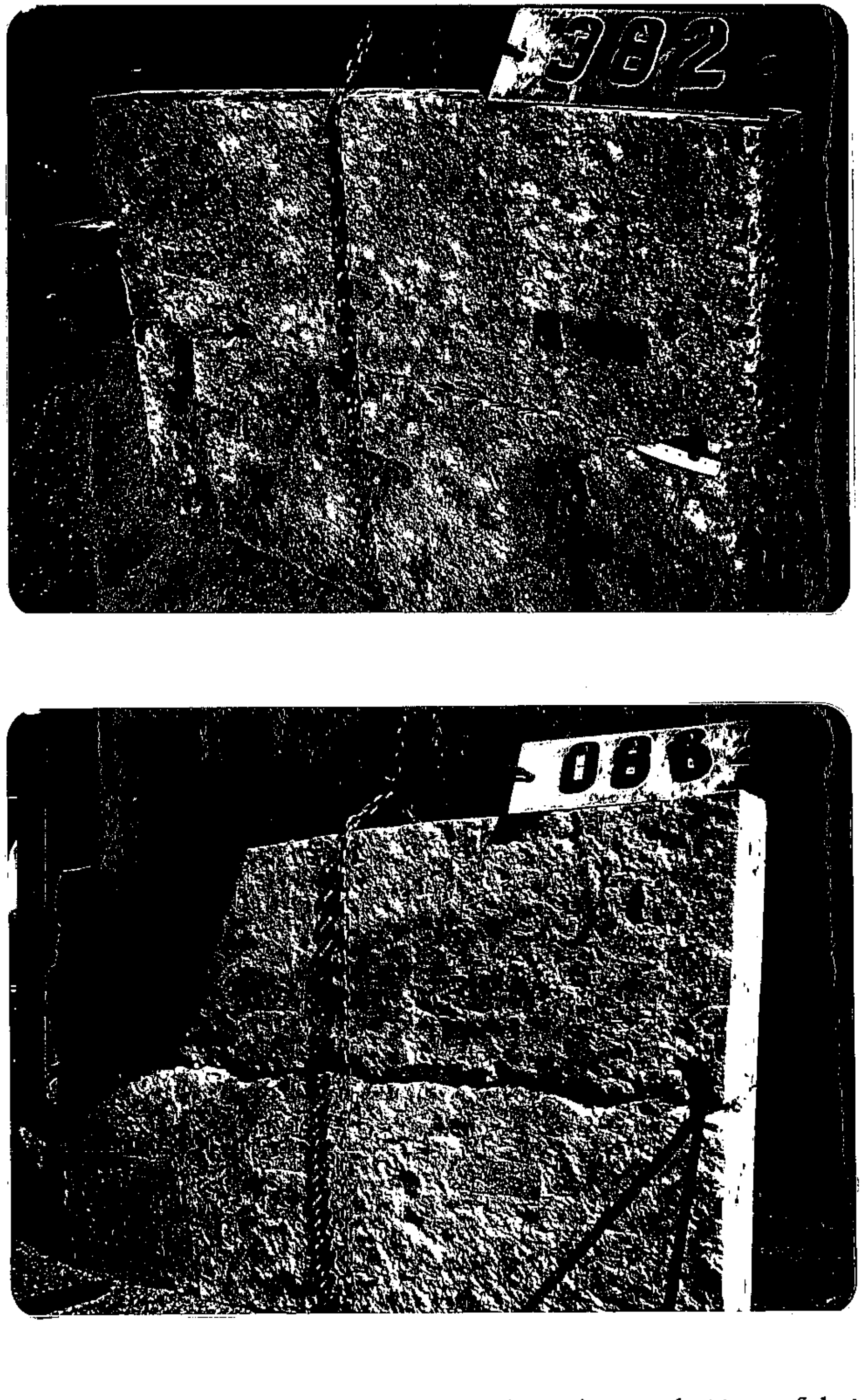


Figure 14. Photographs showing deterioration on bottom of both joints.

that both the surface and the underside of new section 086 had more deterioration than AASHO section 382.

When the concrete was broken away from the bars on one side of the joints with an air hammer, the dowel bar alignment in the horizontal plane and in the perpendicular plane was found to be exact for each half of both joints. The transverse sawcut, however, was 1/2 in. (12.7 mm) off the center of the dowel bar assembly toward the upstream traffic end at both joints.

The decrease in dowel bar diameter caused by corrosion was measured at six places around each bar. The reduction of cross-sectional area averaged 15 percent for both joints. The loss ranged from 9 percent to 25 percent, and both extremes were in bars from section 086. The deepest single depression was 0.22 in. (5.5 mm) in a bar from section 086.

When originally placed, the bars in section 382 were coated with two applications of coal-tar base mill coating, and the bars in section 086 were coated with one application of heavy cup grease. There was nothing on the bars to indicate that any of the original protective coating remained except for very dark non-corroded areas that might have been residue of the original coating.

Top and bottom views of the bars are shown in Figures 15 and 16. Corrosion was limited to a 6-in. band around the bars centered on the joint crack.

To determine crack penetration in the slab, the top surface, 9 in. (230 mm) each side of the joint, was stained with dye. In section 086 the dye penetrated 5 in. (127 mm) deep and 6 in. (150 mm) to 8 in. (200 mm) away from the joint as compared to a penetration of 1 in. (25 mm) in section 382.

When removing the concrete from the dowels, much less energy was required to break up the concrete in section 086 than in section 382. As the concrete was being broken away from the dowel bars, up to three times more aggregate came free of the matrix in section 086 than in section 382. In section 086 a horizontal crack

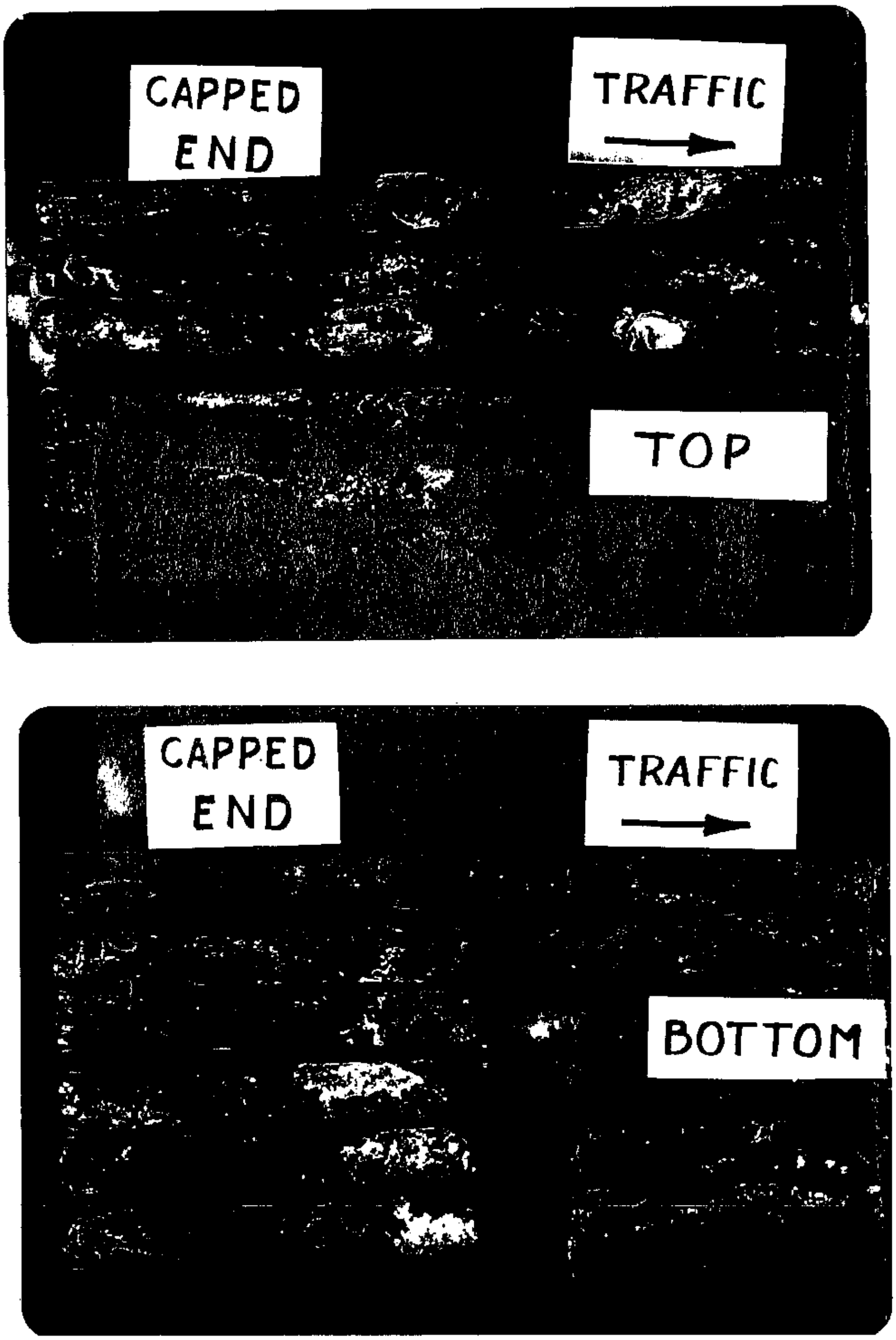
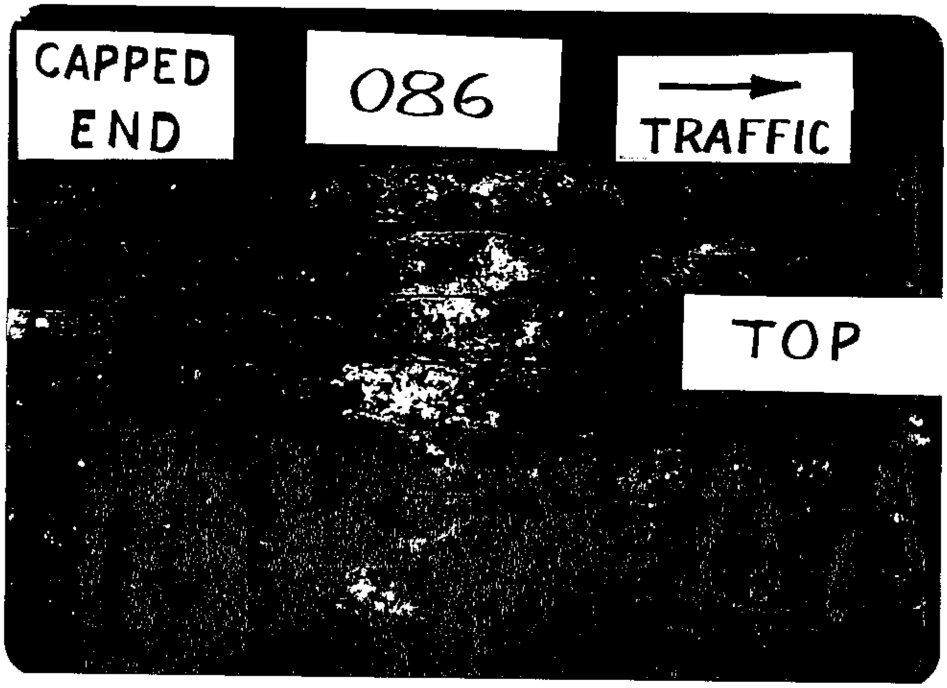


Figure 15. Photographs of dowel bar corrosion in section 382.



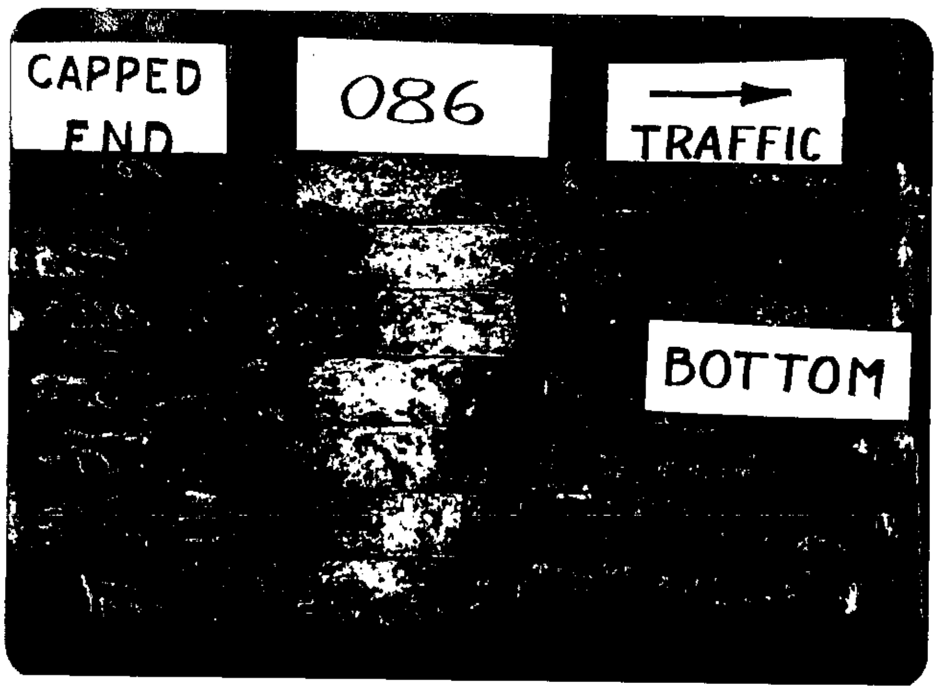


Figure 16. Photographs of dowel bar corrosion in section 086.

had developed in the plane of the dowel bars, and was evident by the manner in which the concrete broke exposing the stained surface (Figure 17). Near some of the bars in section 382, a stain was evident, indicating a horizontal crack was starting to form but not nearly to the extent as seen in section 086.

Concrete Tests

The physical tests performed on the concrete were compressive strength and freeze-thaw durability.

To make compressive strength measurements, cores were cut from part of the pavement removed with the joint. Three cores from each joint were tested with the results tabulated below:

COMPRESSIVE STRENGTH

Section No.	Core No.	psi
382	1	8955
	2	8525
	3	8385
086	1	7740
	2	6680
	3	7085

1 psi = 6.894 kPa

The average compressive strength was 8622 psi (59.4 MPa) for section 382 and 7168 psi (49.4 MPa) for section 086. This 1454 psi (10.0 MPa) difference represents approximately 20 percent higher strength in section 382 than in section 086.

Freeze-thaw tests were performed on beams measuring 3 in. (75 mm) x 4 in. (100 mm) x 16 in. (400 mm) cut from part of the pavement removed with each joint. The tests were performed in accordance with ASTM C 666 Procedure B.

Normally, freeze-thaw tests are run for 300 cycles or until the dynamic modulus drops below 60 percent. The dynamic modulus for beams 086-1, 086-2, and 382-3 dropped below the 60 percent level before 300 cycles. Beam 086-3 had a dynamic modulus of 67.3 percent on the 262nd cycle but broke in two pieces by the 293rd cycle, which was the next time the beams were measured. Beams 382-1 and

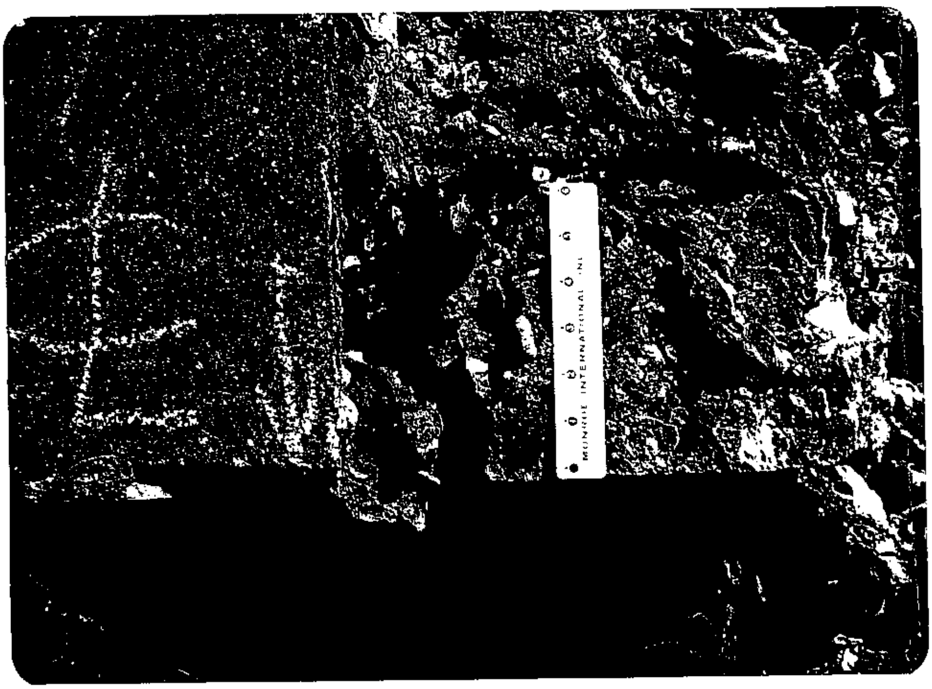


Figure 17. Photograph of crack face in plane of dowel bars, in section 086.

382-2 had a very high modulus on the 293rd cycle, which was the closest number of cycles to 300 at which the beams were measured.

While 300 cycles is the normal length of testing, the beams that did not break in two pieces were tested beyond that point as long as reasonable readings were obtained. A plot of relative dynamic modulus versus cycles, for the entire period of testing, is seen in Figure 18. The smoothed curves represent each beam as marked. Beam 382-2 survived 500 cycles before dropping below a dynamic modulus of 60 percent and beam 382-1 was still above the 60 percent level when testing was stopped at 612 cycles.

Joint Pullout Test

Part of each joint, which was sent to the University of Illinois, was tested for joint lockup by measuring the force required to pull the joint apart. The test was reported in detail elsewhere (12), and is only briefly summarized here.

The pullout test was made with the slabs in a vertical position. This eliminated all friction except that between the dowels and the concrete. The dead load was subtracted from the maximum total load to determine the load required for pullout. A plot of the total axial tensile load minus the dead load versus the joint opening for each joint is shown in Figure 19. Since the maximum total load exceeded the dead load, the joints were definitely locked. The pullout load per dowel for section 382 was 4667 lbs (20.7 kN) and the pullout load per dowel for section 086 was 2420 lbs (10.7 kN). These were the loads required to open the joints 0.04 in. (1.02 mm), which is less than normal opening for joints spaced at 40 ft (12.19 m). These loads convert to a tensile stress of 41 psi (283 kPa) and 21 psi (145 kPa) for section 382 and section 086, respectively. This is not enough to cause cracking in the concrete. However, when combined with

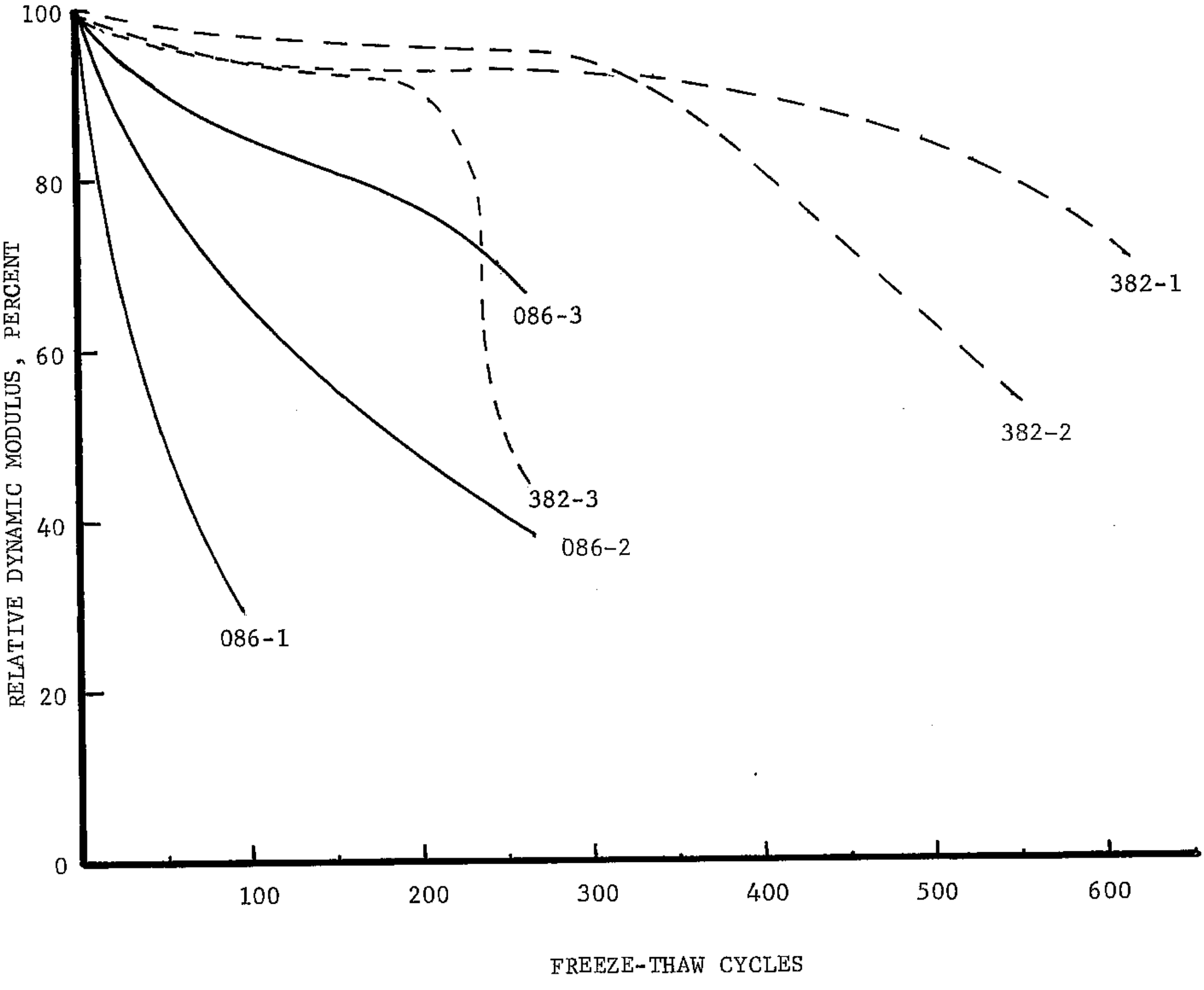
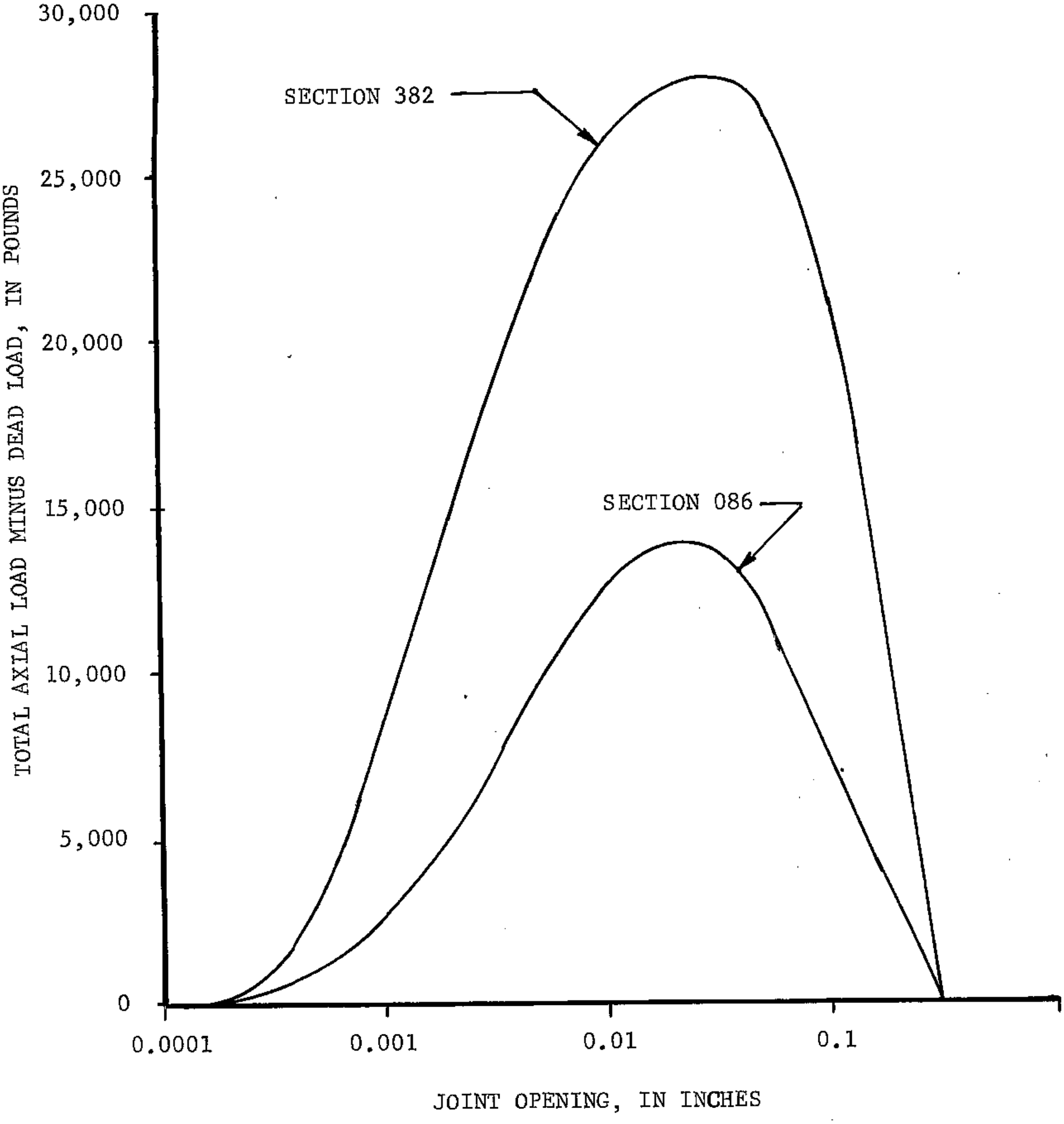
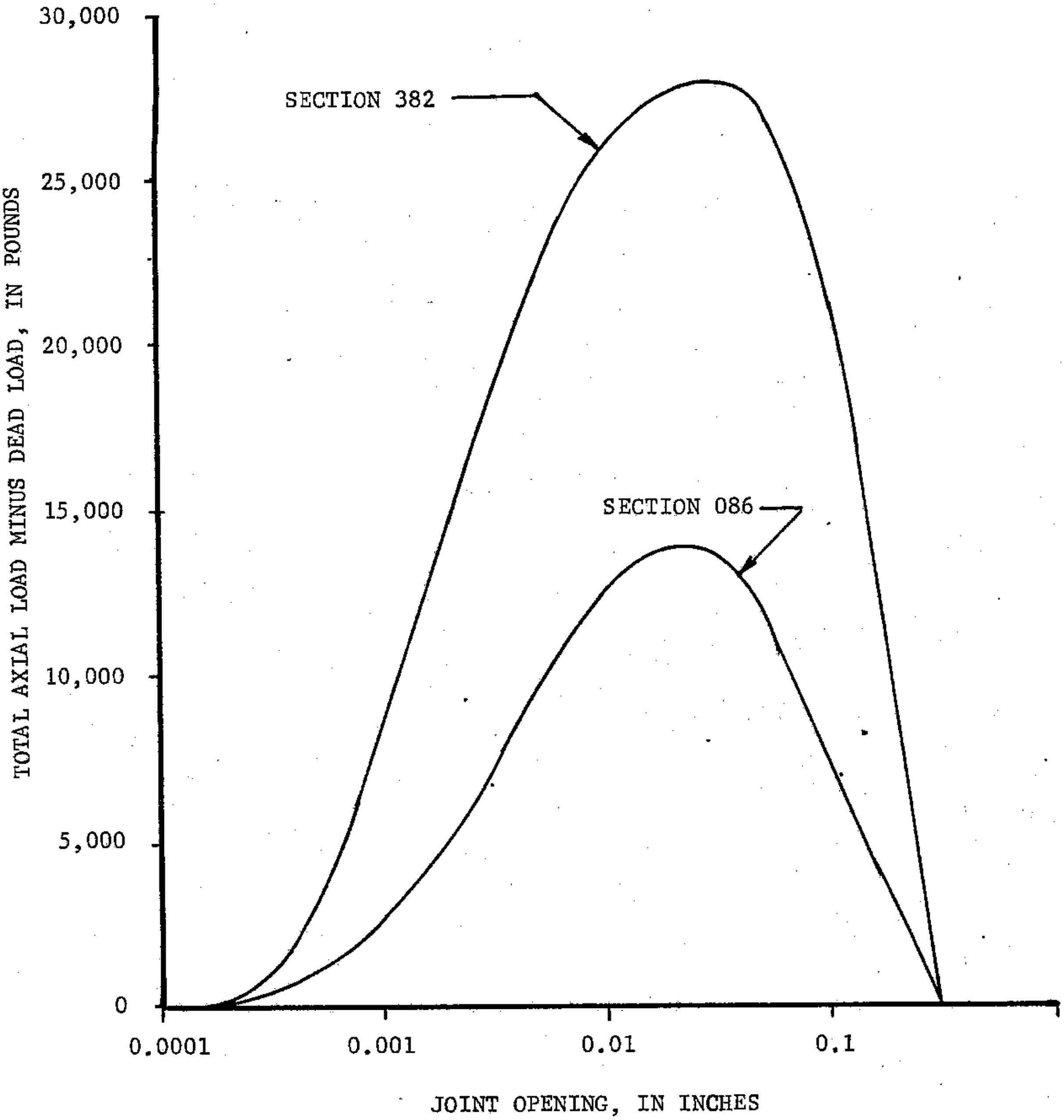


Figure 18. The effect of freeze-thaw cycles on relative dynamic modulus of concrete near a contraction joint.



Note: 1 in. = 25.4 mm; 1 lbf = 4.448 N

Figure 19. Plot of total axial load minus dead load versus joint opening.



Note: 1 in. = 25.4 mm; 1 lb.f = 4.448 N

Figure 19. Plot of total axial load minus dead load versus joint opening.

warping stresses and load stresses, cracking could be initiated. In fact, the panels on each side of the joint in section 382 contained Class 3 cracks, which were faulted, indicating that the steel fabric had ruptured.

One joint was pulled completely apart, and one of the bars was out of alignment. The misalignment of this bar was in the horizontal plane of the dowels, with the end of the bar above the rest.

DISCUSSION

The research described in this report indicates the effect of varying the interval between transverse joints in portland cement concrete pavements on the overall amount of pavement damage that occurred and on the behavior of pavement panels with respect to the level of service that prevailed during the life of the pavements. All the pavement sections had sawed, dowelled, contraction joints with either 15-ft, 40-ft, or 100-ft intervals. Expansion joints were not used.

The portland cement concrete pavement in the AASHO test sections was 8.0 in. (203.2 mm), 9.5 in. (241.3 mm), 11.0 in. (279.4 mm) or 12.5 in. (317.5 mm) thick. The sections with 15-ft panels were nonreinforced and the sections with 40-ft panels were reinforced with welded wire fabric. The pavement sections built in 1962 had a joint interval of either 40 ft (12.19 m) or 100 ft (30.48 m). They were also wire fabric reinforced, and a 40-ft joint interval was used in the sections that were 8.0 in. (203.2 mm) or 9.5 in. (241.3 mm) thick, and a 100-ft joint interval was used in the pavement sections that were 10 in. (254.0 mm) thick.

The principal types of pavement distress that occurred at the transverse joints were spalling, faulting, joint "lockup", and D-cracking. Spalling and D-cracking both resulted in a loss of concrete at joint faces, which ultimately resulted in the necessity for bituminous skin patching.

The faulting that occurred at the joints contributed to pavement roughness. The magnitude of faulting at a joint increased as the interval between joints increased. The use of a stabilized subbase reduced the magnitude of faulting for a given pavement design. The mean fault in the 15-ft joint interval was 1/16 in. (1.6 mm), in the 40-ft joint interval it was 3/32 in. (2.4 mm), and in the 100-ft joint interval it was 1/18 in. (3.2 mm). Although faulting increased with joint interval, the cumulative amount of faulting per unit pavement length was still greatest for the shortest joint interval.

A study of winter joint openings showed that many of the joints were inoperative. Although the fewest number of inoperative joints occurred in the pavement sections with the 100-ft joints, about 25 percent of the joints in the pavement on a granular subbase were inoperative. The 100-ft pavement panels on the BAM subbase had no inoperative joints and the joint openings were very uniform. On the CAM subbase, the joint openings were only slightly less uniform than on the BAM, and all the joints were operating.

The joint pullout tests indicated that frozen joints were most likely due to dowel bar corrosion and not to misalignment. The dowel bar examination showed that both the coal-tar base mill coating and the cup grease failed to provide adequate long-term protection against corrosion.

The tests also suggested that tension stresses, generated as cooling occurred in the pavement slabs with frozen joints, could have formed major cracks by rupturing the steel fabric. The relatively large number of Class 3 cracks that developed in the pavement sections with the greatest number of inoperative joints is taken as evidence that many of the major cracks could have formed in this manner. The comparison of the number of transverse cracks per pavement panel to the mean joint opening in Table 10 shows a definite relation between the number of major cracks that formed and the mean winter joint opening.

Pavement panel behavior was studied by observing the amount of uncontrolled transverse cracking that occurred in the pavement panels as related to joint interval. The results in Tables 8 and 10 showed that very few transverse cracks formed in the 15-ft nonreinforced pavement panels. In the reinforced pavements, the number of minor and major cracks per pavement panel increased as the joint interval was increased. The data in the tables also indicate that the amount of cracking that occurred in a given pavement design was greatest on the granular subbase, intermediate on the CAM subbase, and least on the BAM subbase. It is postulated that the pavement slabs on the stabilized subbases cracked less because they were able to adjust more readily to the internal stresses generated by thermal length changes. This difference in pavement behavior is associated with the nature of the pavement-subbase interface. The BAM subbase is believed to offer the least resistance to pavement movement.

It should be noted that the 15-ft pavement panels were not entirely free of transverse cracks although cracking was much reduced. The cracks in the 8.0 in. nonreinforced pavements all had formed since 1969. The cracks in the 9.5 in. nonreinforced pavements all were present in 1962; however, some have increased in severity. The cracks that formed in the 15-ft panels obviously have not formed as a result of normal shrinkage and hardening of concrete but as a result of either joint lockup or pavement fatigue.

The Roughness Index comparisons (Figure 11 and 12) demonstrate the effect of joint interval on change in riding quality of the experimental pavement sections. As can be seen in Figure 11, the change in riding quality of the AASHO test sections with the 40-ft pavement panels was consistently smaller than the change in riding quality of the sections composed of 15-ft panels. As shown in

TABLE 10. COMPARISON OF MEAN NUMBER OF TRANSVERSE CRACKS PER SLAB TO MEAN JOINT WIDTH

Pavement Thickness (in.)	Subbase Type	Joint Interval (ft)	Mean Number of Cracks per Slab All	Mean Number of Cracks per Slab Major	Mean Winter Joint Width 1/16ths in.
		Original AASHO Test Sections			
8.0	Granular	15	0.3	0.3	1
9.5	Granular	15	0.06	0.04	1
11.0	Granular	15	0	0	2
12.5	Granular	15	0	0	2
8.0	Granular	40	3.0	1.6	1
9.5	Granular	40	2.3	1.03	2
11.0	Granular	40	2.1	0.7	3
12.5	Granular	40	1.8	0.3	3
		New Test Sections			
8.0	Granular	40	2.2	0.7	0
8.0	CAM	40	1.3	0.3	1
8.0	BAM	40	1.3	0	0
9.5	Granular	40	1.4	0.9	0
9.5	CAM	40	1.6	0.5	2
9.5	BAM	40	1.0	0.7	0
10.0	Granular	100	6.4	0.6	2
10.0	CAM	100	3.9	0.2	4
10.0	BAM	100	2.1	0	5

Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m

Figure 12, the change in riding quality of both the 9.5-in. and 10.0-in. pavements was reduced by the use of a stabilized subbase.

Increase in pavement roughness and loss in pavement serviceability was definitely associated with the number of transverse joints per unit length of pavement.

CONCLUSIONS

1. The cumulative amount of pavement damage and the maintenance requirement per unit length of pavement increased directly as the number of joints per mile increased.
2. The amount of damage per joint increased as the joint interval increased.
3. Faulting decreased as slab thickness increased and increased as the joint interval increased. Faulting was reduced on the stabilized subbases.
4. The cumulative amount of faulting per pavement mile is greatest on short panel lengths even though they fault less.
5. Winter joint openings in the 100-ft pavement panels on a BAM subbase were large enough to prevent corrosion from freezing the load-transfer dowels in place.
6. The best level of service throughout the life of the experimental rigid pavements was associated with the longest joint interval.
7. Uncontrolled transverse cracking was almost eliminated in the 15-ft pavement panels.
8. The formation of major panel cracks was associated with inoperative joints.
9. Dowel bar corrosion, rather than dowel misalignment, was the major cause of inoperative joints.
10. Formation of transverse cracks was reduced in the pavement panels on the stabilized subbases. The reduction was greatest in the 100-ft panels on the BAM subbase.

IMPLEMENTATION

The conclusions drawn from this study were based on an evaluation of twelve years of performance data on the joints included in the rehabilitated AASHO Test Road project. The joint spacing was 15 ft (4.57 m) for nonreinforced pavement and 40 ft (12.19 m) for reinforced pavement in the original AASHO Test Road and 40 ft (12.19 m) and 100 ft (30.48 m) for reinforced pavements in the new pavement. The 100-ft joint spacing was the standard in Illinois and several other States during the study period and has remained so for a number of years. The conclusions indicate that the longer 100-ft joint spacing provided the best overall pavement performance. While the amount and severity of both spalling and faulting per joint increased, the accumulative effects of these per mile of pavement decreased as the joint interval increased. The amount of faulting per mile of pavement, which will ultimately require maintenance, decreased as the joint interval increased. Also, the loss in riding quality with time and traffic tended to decrease as the joint interval increased. The results tend to substantiate the old theory that problems with conventionally reinforced PCC pavements are at the joints, and the fewer joints one puts in, the less problems one has.

In considering implementation of the findings, it is necessary to look at the limitations placed on these findings by the design of the joints under study in combination with advancement in technology that has been made in the area. All joints included in the study were formed by sawing a 1/8-in. wide groove. The sealing of the joints was accomplished with a cold-applied rubber-asphalt material in the 1/8-in. groove. None of the joints remained sealed, which allowed incompressibles to enter. Since the initiation of this study, considerable advancement has been made in joint-sealant materials and in the shape factor of the reservoir to hold these materials. The concept also has been advanced that joint design spacing should be compatible with the material used for sealing the joints and with the environmental conditions under which the joints will serve.

Thus, for implementation purposes, the findings of this study are being interpreted as indicating that transverse contraction joints should be spaced the maximum distance possible that will still be compatible with the type of sealant used and the anticipated maximum opening that will occur during the cold winter months. The findings from this study, combined with information from other sources (13, 14), have resulted in the following changes in the Illinois Standard Design for contraction joints in PCC pavements.

(1) Require plastic-coated dowel bars for load transfer at joints.

(2) Seal all contraction joints with neoprene compression seals.

(3) Reduce the spacing between contraction joints from 100 ft (30.48 m) to 50 ft (15.24 m).

(4) Reduce the size of welded wire fabric and bar mat reinforcement to correspond with the reduced joint spacing.

(5) Reinforce only the center 35 ft (10.67 m) of pavement between contraction joints.

(6) Revise the specifications for construction joints to require deformed tie bars in lieu of load-transfer dowel bars.

Relative to the first change, the findings from this study reinforced other findings that corrosion at dowel bars can cause joint lockup and prevent the joints from performing satisfactorily during pavement thermal length changes. This was demonstrated in the detailed examination of two joints and by the fact that many of the joints in later years failed to open during the cold winter months. Plastic coatings have been shown to prevent corrosion of dowel bars. The use of plastic coating also eliminates the need for the heavy greasing which, on occasion, has caused early faulting at joints due to voids being formed between the dowel bar and the concrete. The application of a light coating of form oil has been shown to prevent bonding of the plastic coating to the concrete.

Relative to the second change, all literature and available product information indicate that the preformed neoprene compression seal is the best and most effective sealant presently available for contraction joints in the new PCC pavement. This type of sealant can be compressed up to 50 percent of its uncompressed width, and should be maintained at least 20 percent compressed throughout its life.

In the third revision, 50 ft (15.24 m) was selected as the maximum spacing between contraction joints that would be compatible with the neoprene compression-type seal with the environmental conditions that exist in Illinois, and at the same time would not adversely affect the as-constructed riding quality of new pavements. To maintain the 100-ft joint spacings it would require a one-inch-wide reservoir and a two-inch-wide compression seal. This would permit a maximum joint opening of 0.6 in. (15.2 mm) in the winter months, which would be compatible with Illinois environmental conditions but would adversely affect the riding quality. A 1.6-in. joint opening would ride as an expansion joint and would be felt by the traveling public. The 50-ft spacing requires an initial reservoir width of 0.63 in. (16 mm) and a neoprene compression seal 1.25 in. (32 mm) wide. This will accommodate a maximum opening of 0.38 in. (10 mm) in the winter time and still maintain 20 percent compression in the seal.

The fourth change is a natural outgrowth of the adopted closer joint spacing. Pavement reinforcement is required to hold transverse panel cracks tightly together to provide load transfer by aggregate interlock. With shorter panels, the amount of reinforcement needed for this purpose is reduced. Procedures developed by PCA were used to determine the weight of reinforcement for 50-ft panels.

The fifth change, to reinforce only the center 35 ft (10.67) of pavement between contraction joints or 70 percent of the pavement area, was adopted as a cost-reduction measure. The cost savings involved in lighter reinforcement and in utilizing only 70 percent reinforcement in the pavement area would partially offset the added cost of doubling the number of contraction joints, dowel bar assemblies, and the addition of the neoprene compression seal. Data collected from this study, from other studies of pavement condition in Illinois, and from some work done by Minnesota show that cracks in PCC pavements mostly develop within the middle one third of a panel and rarely occur outside the center two thirds of a panel. Thus, since the steel reinforcement serves only to hold panel cracks together, it serves no useful purpose in the areas immediately adjacent to contraction joints where transverse cracks do not develop.

The final change, to require deformed tie bars in lieu of load-transfer dowel bars at construction joints, was adopted for two reasons. First, it permits maintaining a uniform 50-ft spacing between contraction joints without placing an undue harship on construction to place headers at the end of a day's paving exactly at the location of a contraction joint. Secondly, standard practice has been to place loose dowel bars through drilled headers at the end of a day of paving. This has often resulted in misalignment of the individual dowel bars, and the improper functioning of the construction joint as a contraction joint. By tying construction joints with deformed reinforcement bars in a quantity sufficient to withstand shear stresses and by not edging the joint, it will be unnecessary to seal construction joints and should eliminate the problems that have developed at these joints in the past.

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APPENDIX A

Layout of Test Sections

In Figure A-1 and A-2, the symbol identified as BC-Base Course is the crushed stone-special which was the original base material in the AASHO Test Road. Test sections 016 and 018, which have the same symbol, represent salvaged crushed stone-special placed when the pavement was rehabilitated.

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9/12/78- Helen took these original Figure pages Red No.'d 86 thru 94 to put in P.R Report No. 76.

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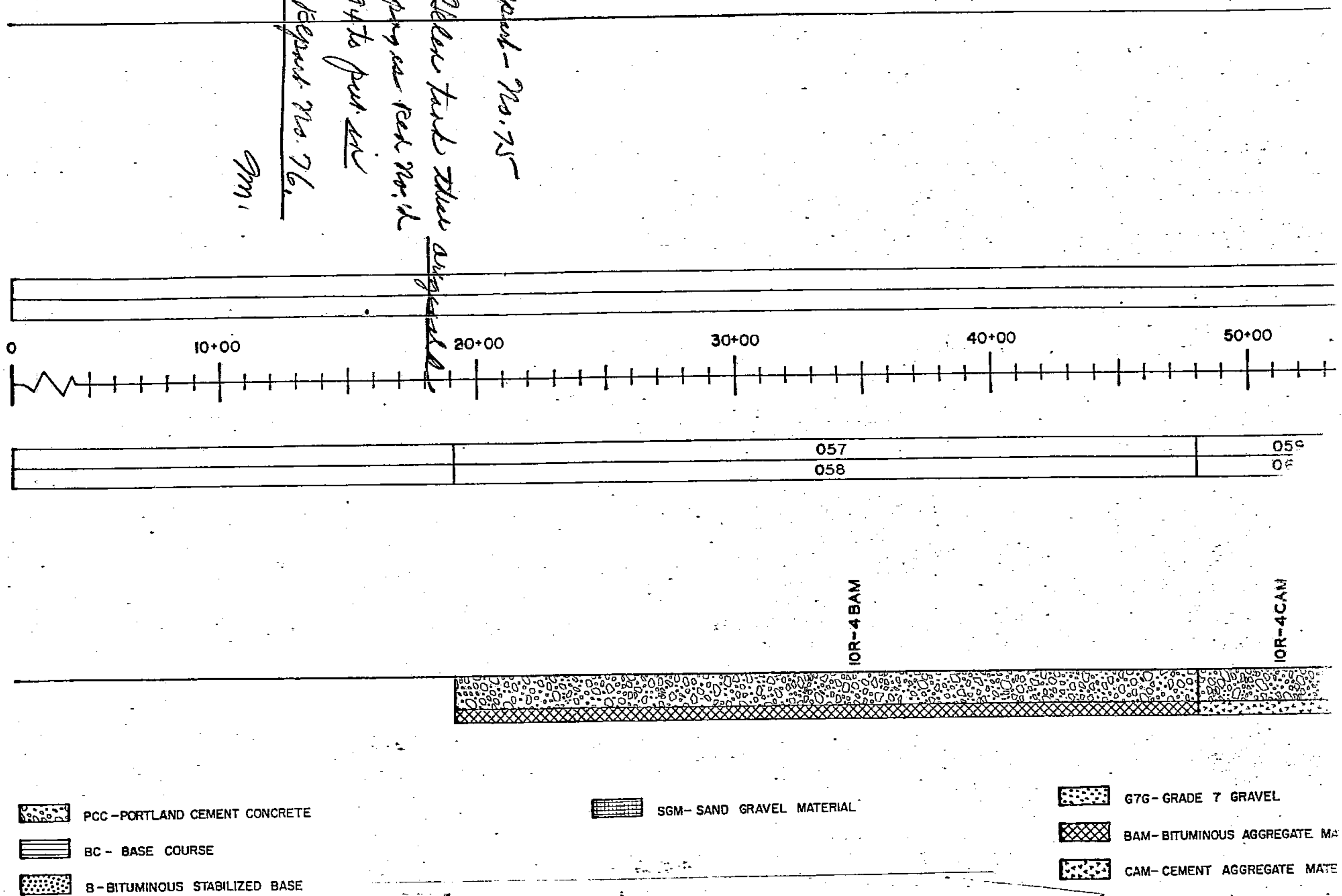
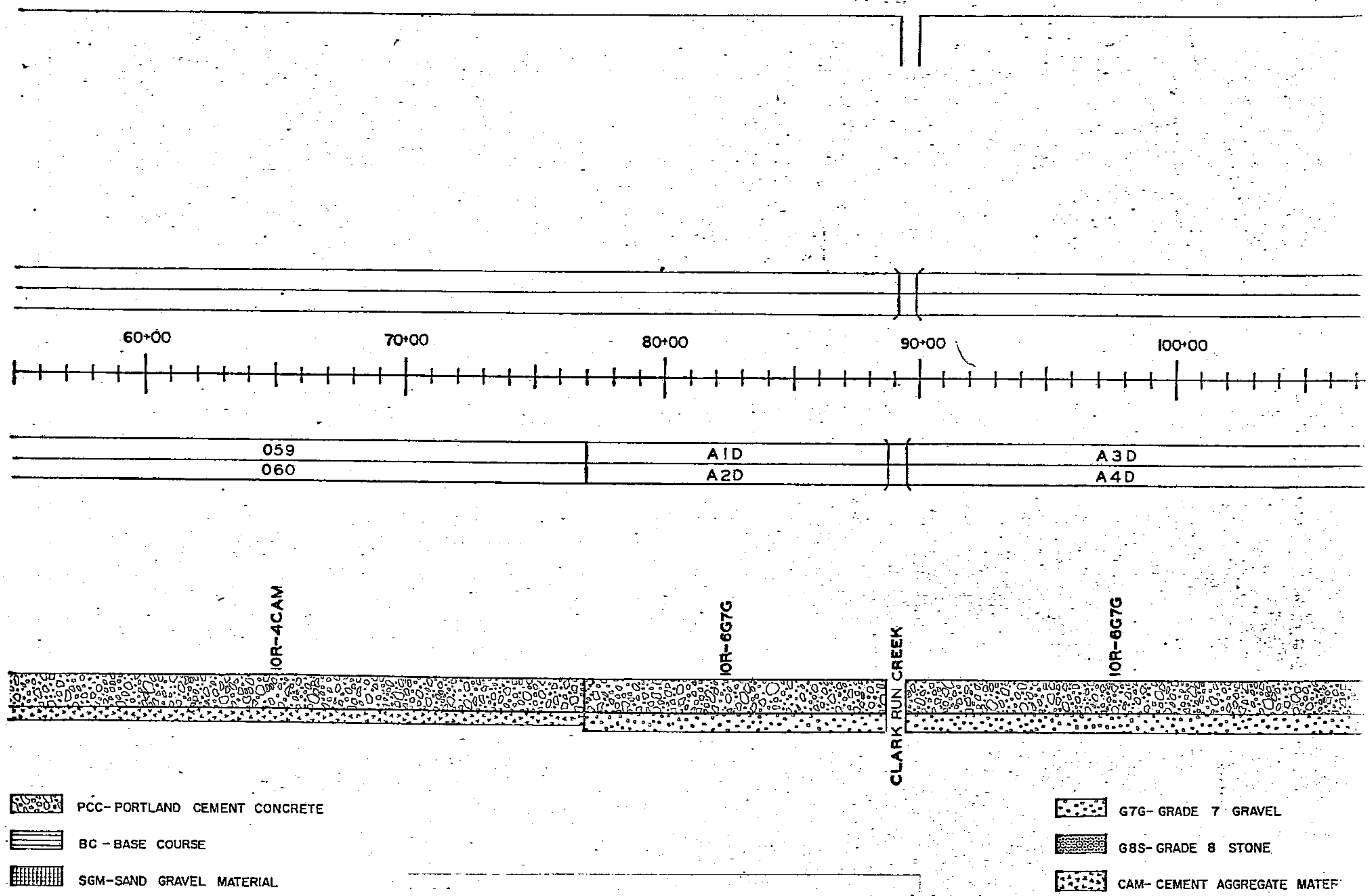


Figure A-1. Layout of test sections.



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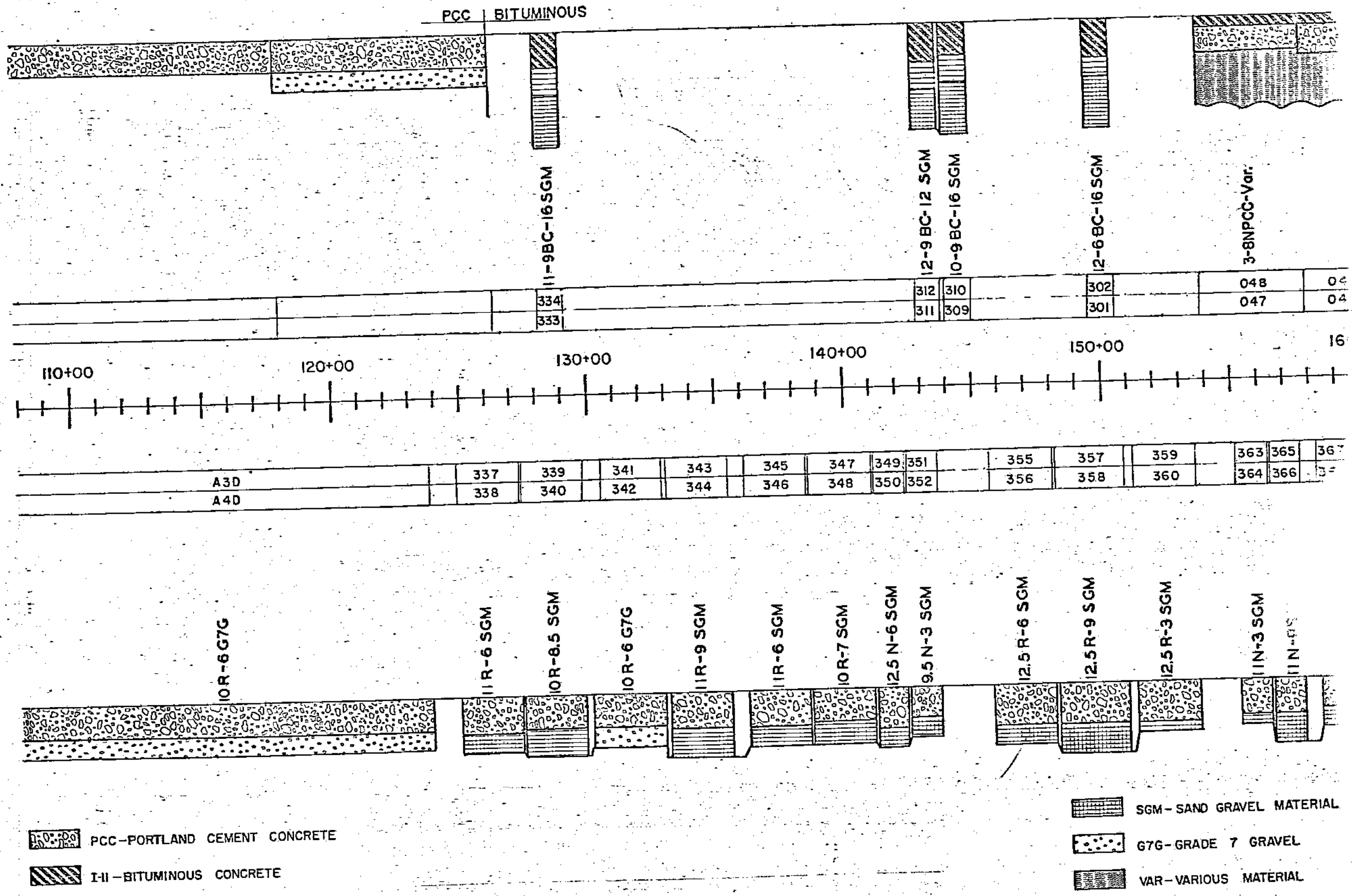

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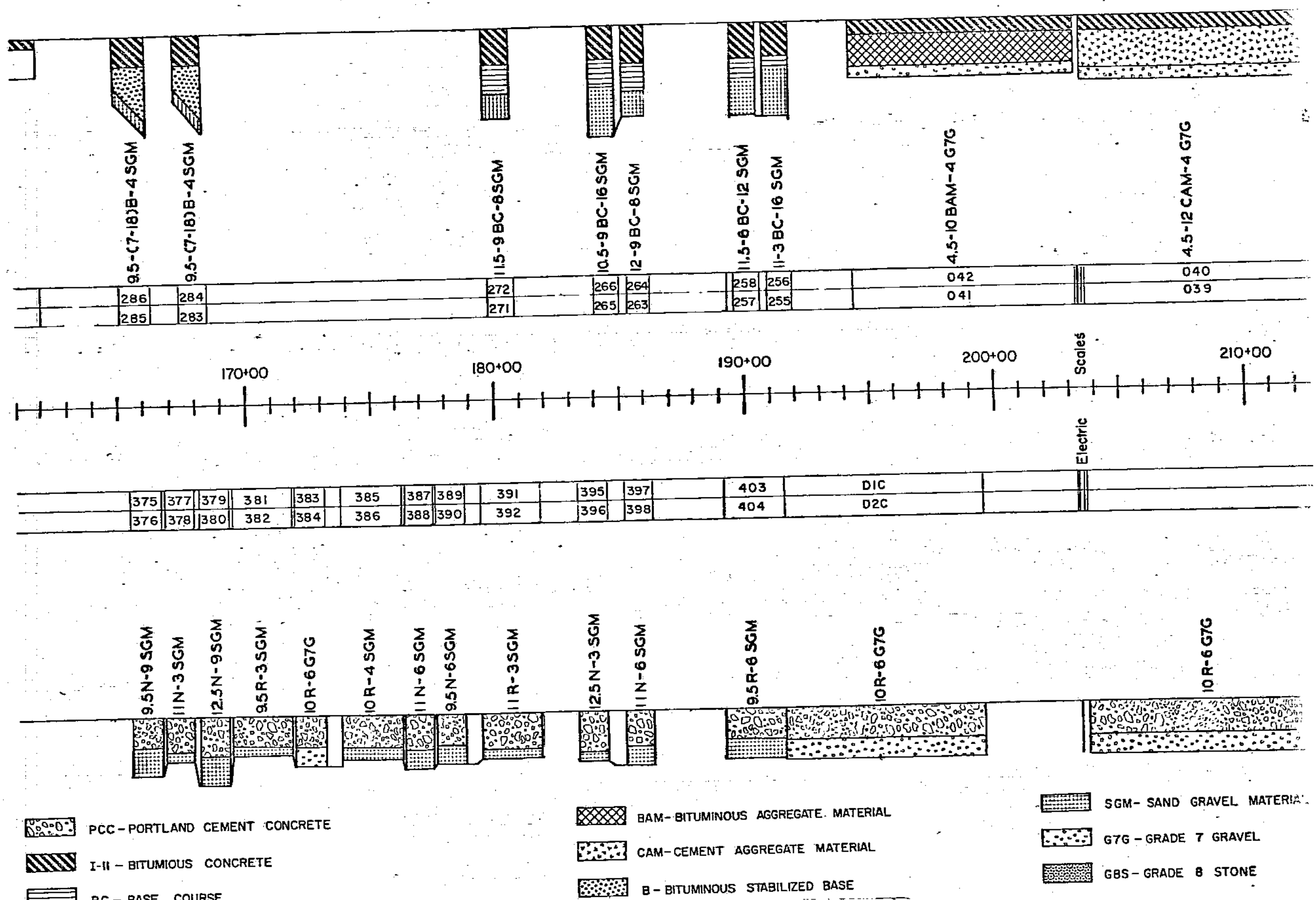

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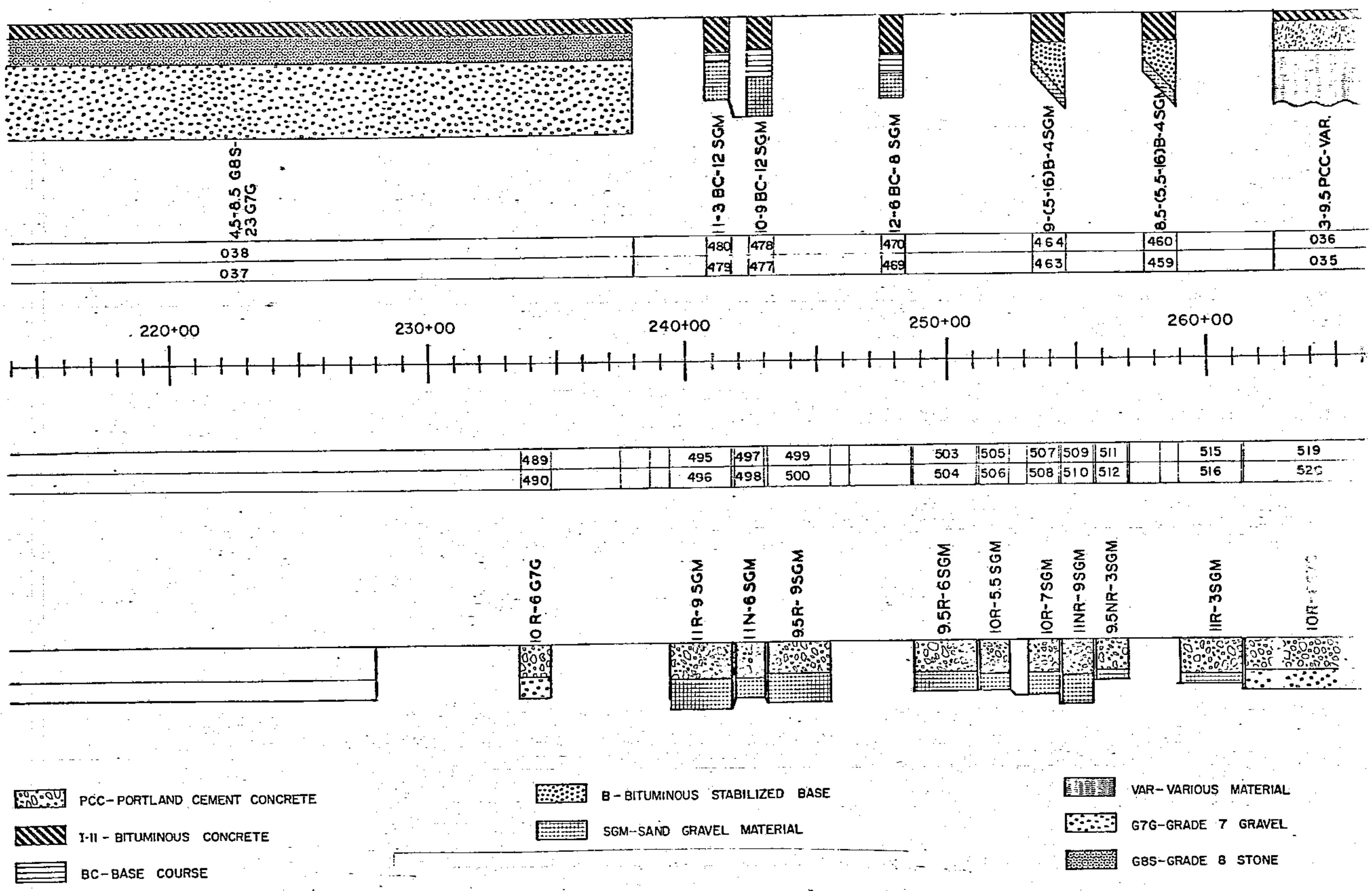

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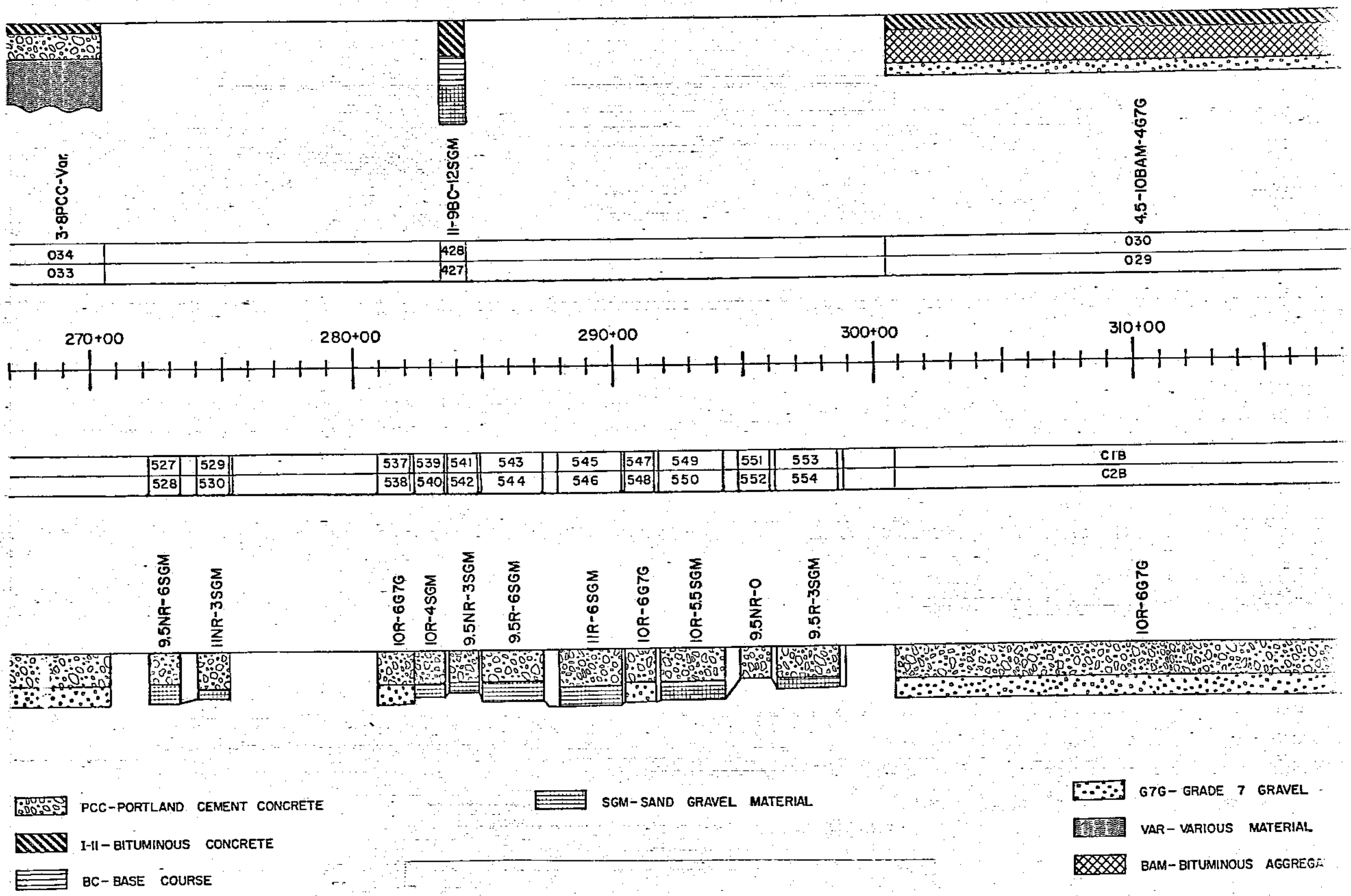

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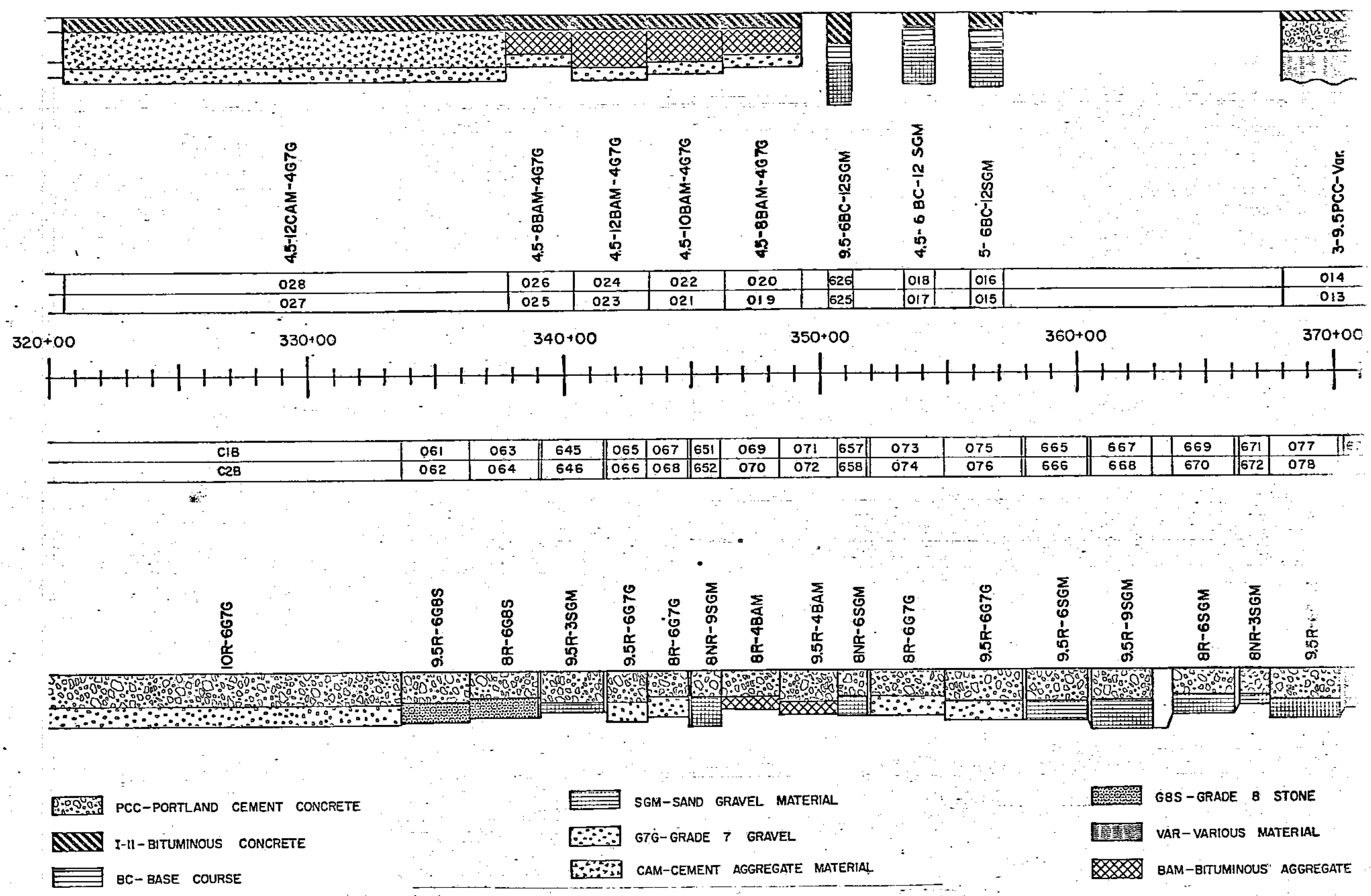

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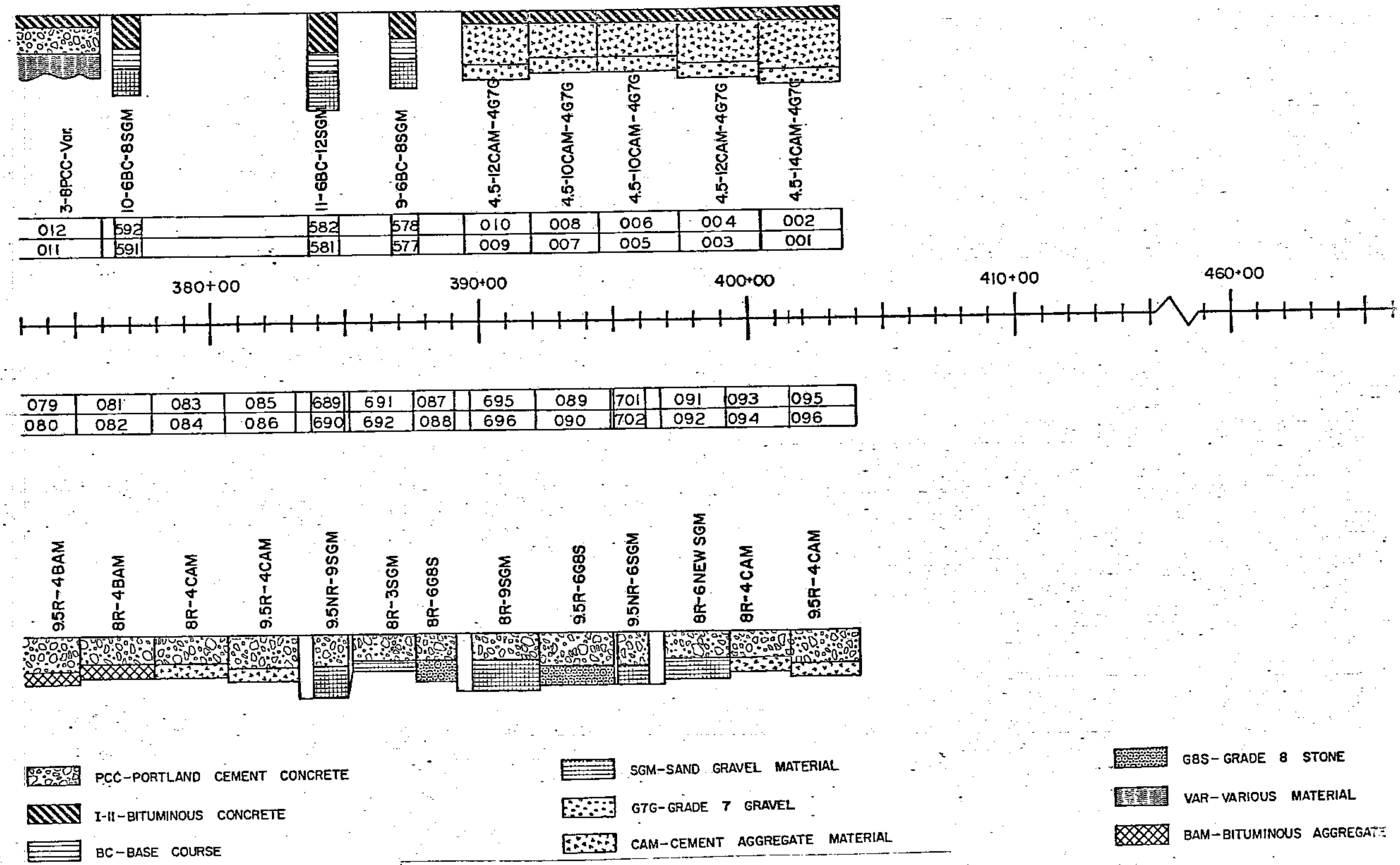

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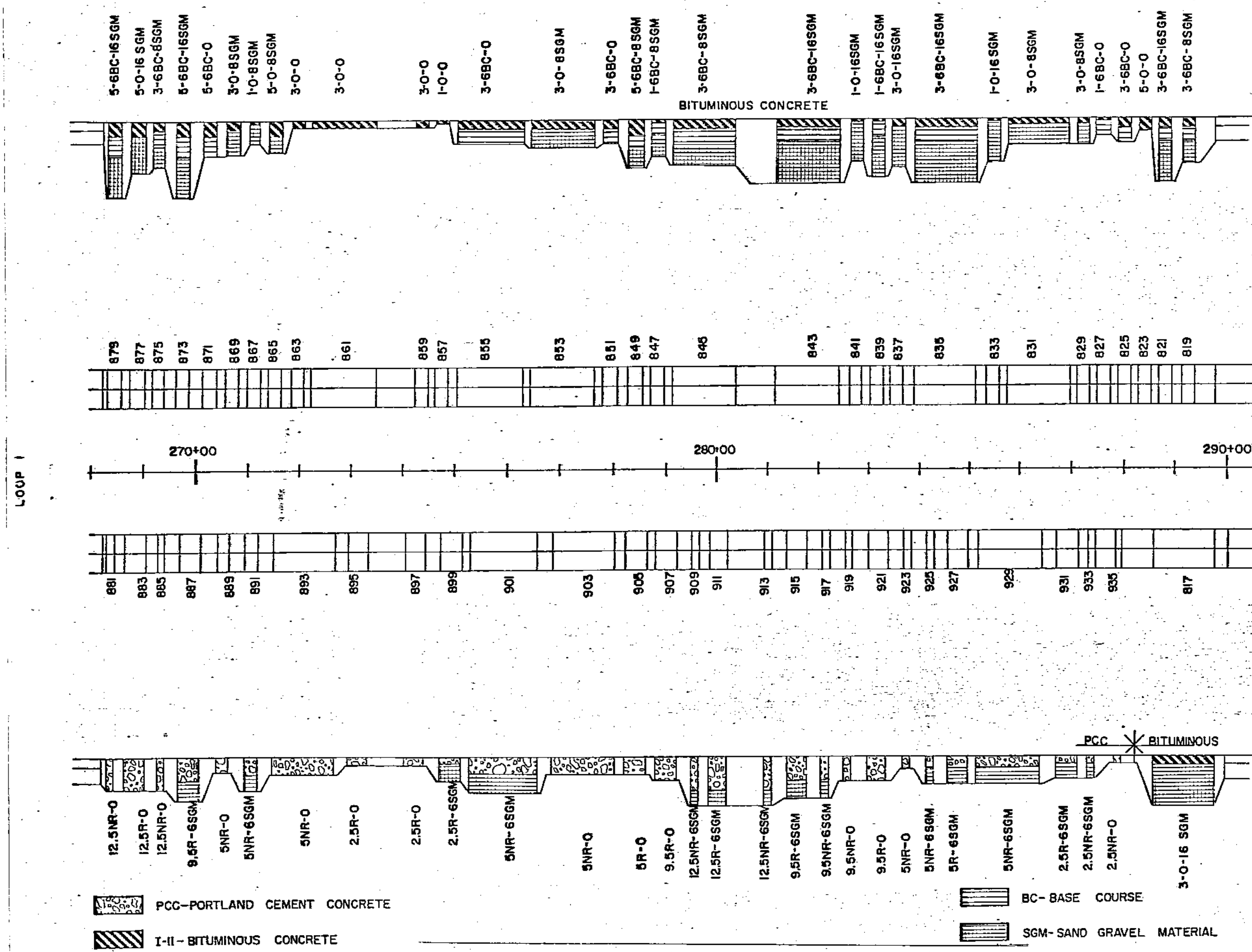

Figure A-2. Layout of test sections - Loop 1.